STUDIES ON REINFORCED FOUNDATION BEDS AND DOUBLE FACED VERTICAL WALLS.

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CERTIFICATE

This is to certify that the thesis entitled "STUDIES ON REINFORCED FOUNDATION BEDS AND DOUBLE FACED VERTICAL WALLS" by Trilochan Sahu, 9010329 , is a record of work carried out by him under my supervison and has not been submitted elsewhere for a degree.

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CONTENTS

ABSTRACT

In this thesis, an attempt has been made to make a parametric study of the bearing and settlement aspects of foundations on reinforced beds. A compilation of many of the available studies on reinforced foundation beds have ben made. The results are th~~us~~en analysed to find the effects of different parameters like depth of the top layer of the granular fill, number of reinforcement layers, depth to the top layer of reinforcement, vertical spacing of the reinforcing layers and the length of the reinforcement, on the bearing capacity and settlement response of the reinforced foundation beds.

Based on experimental investigations conducted on a model reinforced double faced wall, the effect of spacing between the reinforcing layers and the length of overlap of the reinforcement have also been studied. The qualitative and quantitative estimation of the improvement in the reinforced bed behaviour have been studied in the thesis. The parametric study reveals the significant influence of the various parameters on their bearing capacity and deformation characteristics.

LIST OF FIGURES

CHAPTER I

INTRODUCTION

Soil reinforcement is one of the fast growing techniques which is becoming popular all over the world. Basically, reinforced soil is a construction material comprised of soil fill strengthened by inclusion of metallic strips, ties or geogrids, geotextiles etc. which frictionally interact with the soil making it a coherent mass.

One of the potential fields of application where reinforced soil can prove to be an effective alternative to the conventional ground improvement techniques is in the improvement of the bearing capacity of weak soils. Reinforced soil bed is a soil foundation containing horizontally embedded metallic flat thin strips or geosynthetics (geotextiles, geogrids, geomembranes etc.). As the reinforcing action requires good frictional bond between the grids and the soil, granular fills are invariably provided; the system may be termed as a reinforced granular slab or bed. The strengthening of the soft soil by reinforced slabs lead to an increase in stiffness with a consequent decrease of settlement and increase in the bearing capacity.

Binqueet and Lee (1975) were the first to report a systematic study on reinforced foundation beds, from their experimental observations and subsequent modelling of the system. Since, then a large number of analytical and experimental works have been conducted by many researchers to find out the failure mechanism

and the load settlement behaviour of the reinforcd foundation bed, so that a suitable design method can be evolved for reinforced granular beds. But as the experiments have been conducted on locally available materials, there is a large variation in the results; however, a general trend of increase in bearing capacity due to the inclusion of reinforcement has been observed in all the cases.

The reinforced earth technique is also adopted in the construction of embankments. Geotextiles and Geogrids can be used to construct a comparatively steep embankment replacing the conventional flat slopes. This allows for a more efficient land use and reduction in the cost of construction. For the design of reinforced embankment, the strength of the reinforcement, the spacing of the reinforcement layer and the overlap between the free ends of the lower layer and the upper layer are important from design considerations.

An attempt has been made in this thesis to make a parametric study of the bearing capacity and settlement aspects of foundations on reinforced beds. Chapter II provides a brief literature review on the experimental and analytical studies made by various researchers on the bearing capacity of reinforced soil beds.

Chapter III gives a detailed compilation of many of the available studies on reinforced foundation beds and the results are analysed to find the effects of different parameters (depth of the top layer of the granular fill, number of reinforcement layers, depth to the first layer of reinforcement, vertical

spacing of the reinforcement layer and length of the the reinforcement) on the bearing capacity of the reinforced soil beds.

Chapter IV gives the details of the model studies that were carried out to find the response of a reinforced double faced vertical wall(which can be used as an embankment) varying the spacing between the reinforcing layers and the width of the overlap. A brief discussion of the test results obtained and the scope of future studies haves also been given in this chapter.

CHAPTER II

LITERATURE REVIEW

2.1 INTRODUCTION

Reinforced soil is a composite material comprising of soil fill strengthened by inclusion of metallic strips, ties or geogrids, geotextiles, etc. which frictionally interact with the soil and make it more resistant and less deformable than the soil alone. The technique of soil reinforcement itself is not a new one and has been in vogue since long time for constructing earth walls with different types of reinforcing intrusions like rope-fibre, bamboo strips, straw reeds, branches of trees, etc. The first systematic study on reinforced soil was carried out by Vidal(1965), whose pioneering work has made it possible to use this technique in a number of diverse fields of earthwork construction.

2.2 EXPERIMENTAL STUDIES

One of the potential fields of application of reinforced soil is its use in the improvement of bearing capacity of soft or weak soil, using it as a reinforced soil bed below the structural foundation .Binquet and Lee (1975) were the first to report a systematic study on reinforced sand beds. They performed model plate load tests simulating strip footings on reinforced sand beds and made parametric studies.The effects of number of layers and the depth to the top layer of reinforcement were investigated.

Akinmusuru and Akinbolade (1981) performed laboratory model tests with square footings on a deep homogeneous sand bed reinforced with flat strips of the locally available rope fibre material. The effect of horizontal spacing of fibre strips, vertical spacing, the number of layers and the depth to the top most layer of reinforcement on the bearing capacity of the reinforced soil have been studied. They (1985) have also reported the effect of the same parameters on horizontally reinforced layers of crushed rock and compared their results with the previously reported values.

Saran and Talwar (1981) conducted bearing capacity tests on a sand subgrade reinforced with aluminum foil and fibre glass strips. The parameters studied were the depth to the first layer of reinforcement and the strength of the reinforcement .

Patel and Paldas (1981) conducted model tests on a reinforced sand bed with a strip footing to evolve the suitability of different reinforcing fabrics. Patel (1982) studied the effect of the shape of the footing on the performance of the reinforced sand bed. Model tests were carried out on circular, strip, and rectangular footing representing axi-symmetric, plane-strain and restricted 2-D state of stress .

Andraws et al.(1983) studied the load settlement behaviour of a strip footing when a single layer of geotextile is placed at different positions in dense sand. Deformation vector of sand grains and tensile forces in the geotextile have been measured using a photogrammatric technique.

Milovic (1984) filled the gap between the laboratory and the field behaviour by carrying out field load tests with a circular rigid foundation placed on a reinforced soil bed. The reinforced soil bed consisted of a gravel layer, reinforced by steel bars, resting on a soft silty clay. He studied the effects of the thickness of gravel layer and the reinforcement in the gravel layer on the bearing capacity of the system.

Fragaszy and Lawton (1984) studied the effect of soil density and the length of the reinforcement on the load-settlement behaviour of reinforced sand beds reinforced with aluminium foils by carrying out a series of tests with a rectangular footing.

Guido et al (1985,1987) conducted laboratory model tests on a shallow square footing to study the bearing capacity of soils reinforced with geotextiles and geogrids. The parameters studied were the depth to the first layer of reinforcement, number of reinforcing layers, the size and the tensile strength of the geotextile.

Dembicki et al.(1985,1986) carried out tests on a rigid strip foundation resting on a two layer soil, consisting of a layer of geotextile reinforced sand fill on the surface of soft clay. The effect of sand layer, the depth of first layer of reinforcement, number of reinforcing layers, the length of the geotextile and the inclination and eccentricity of the applied load, on the bearing capacity of the two layer system were studied.

Love et al.(1984,1987) studied the ef tiveness of geogrid reinforcement placed at the base of a lay granular fill on

the surface of a soft clay by small scale model tests conducted under plain strain condition using a range of fill thicknesses and subgrade strengths.

Ingold and Miller (1985) carried out model studies with a strip footing resting on London clay reinforced with geogrid. They studied the effect on bearing capacity of the depth to the first layer of reinforcement and the numbers of layers.

Milligan et al.(1986) conducted full scale tests with strip and circular footings on a granular layer over a weak subgrade reinforced with geogrid placed at the interface between the two layers.

Jarret (1986) conducted a large scale plane strain loading test on a gravel fill compacted over a soft deposit. The effects of fill thickness of varying thickness with and without geogrid reinforcement were studied for analysing the tension membrane action of the reinforcement.

Resl and Werner (1986) studied the influence of geotextile on the bearing capacity of subgrade by carrying out plate load tests for various subgrade and fill materials.

Jarret and Bathurst (1986) performed a large scale model test to compare the load deformation behaviour of a gravel in-filled geoweb/geocell mattress and unreinforced gravel bases over peat under plane strain static loading.

Gardiel and Morel (1986) studied the suitability of various textile elements i.e. continuous filaments (Texol),macro-geogrids (Tensar), geotextile cells filled by soil (Armater, Nidaplast) for constructing low volume unpaved roads. They carried out laboratory

model tests with a circular rigid plate to study the load-settlement behaviour and the behaviour under small and large deflections.

Verma and Char (1986) studied the efficacy of vertical reinforcement on improving the bearing capaciity of soil subgrade. Their study included model tests with a strip footing to determine the influence of the extent, spacing and flexibility of the reinforcing elements on the bearing capacity and settlement behaviour of reinforced sand.

Sridharan et al.(1988) carried out model tests with a circular footing on granular fill over a soft layer of saw dust.Parametric studies demonstrate the effect of the thickness of the granular fill with or without reinforcement, numbers of reinforcing layers, their position and the type of reinforcement on the footing behaviour.

Sankariah and Narahari (1988) conducted model footing tests on a sand bed reinforced with three type of reinforcement i.e.GI strips, bamboo strips and mild steel welded mesh. They studied the influence of number of reinforcing layers, the depth to the first layer of reinforcement and the type of reinforcement on the bearing capacity and the settlement performance of the reinforced soil.

Sreekantiah (1988) studied the behaviour of reinforced bed in improving the bearing capacity and settlement response under strip and square footings using aluminum foil as reinforcing elements. Parameters varied were the number of layers and the depth to the top layer of reinforcement.

Das(1988) carried out model studies on a compact granular fill material underlain by a soft clay layer with and without inclusion of a geotextile and studied the beneficial effect of the geotextile on the bearing capacity of the composite layer.

Samtani and Sonpal (1989) investigated the bearing capacity aspects of strip footings on cohesive soil reinforced with metal strips by carrying out model tests and studied the failure mechanism and failure profile in reinforced cohesive soil.

Das(1989) carried out model tests with strip and square shallow foundations supported by a compact sand layer underlain by a soft clay with and without the use of geotextile at the interface.

Chung and Tatsuoka (1990) carried out a series of plain strain model tests with a strip footing resting on a reinforced sand bed. They studied the effect of the length, the arrangement, the rigidity and rupture strength of reinforcement on the bearing capacity of the reinforced soil.

Puri and Das (1990) extended the study of Verma and Char with model studies on vertically reinforced sand beds. They carried out a series of model tests with a strip footing on a sand layer with vertical semi-flexible reinforcement (thin metal strips) and studied the effect of various parameters on the bearing capacity and settlement behaviour. The parameters studied were the relative density of sand, roughness of the rods, the diameter, spacing and extent up to which the rods were placed.

Shimizu and Iuni (1990) carried out model tests on soft ground (loose sand) reinforced by a six sided geotextile cell

wall. They studied the effect of the size of the cell and strength of the geotextile on the bearing capacity of the reinforced soil.

Khey and Perier (1990) conducted model and full scale tests to determine the bearing capacity of a soft silty soil, overlain by a layer of geotextile cells filled with fine grained sand.

Desmukh (1991) carried out model tests with strip, rectangular, square and circular footings on a marine sand reinforced with needle-punched geotextile. The effect of three parameters i.e.location, number and size of the geotextile layers on bearing capacity and settlement characteristics was studied.

2.3 ANALYTICAL STUDIES

Binquet and Lee (1975) proposed an analytical method for design of reinforced soil beds. Based on a model test behaviour three possible modes of bearing capacity failure, (i) shear failure above reinforcement (ii) pullout of ties due to slip and (iii) tension failure of ties, which form the basis for the bearing capacity analysis were considered.

Brown and Poulos (1981) used a finite element model to investigate the increase in bearing capacity and stiffness of a foundation due to placement of reinforcement in the soil. Their analysis showed that the quality of the reinforcement necessary to produce a significant increase in bearing capacity is high. The limit state of the reinforcement-soil bond was reached at an early stage. It therefore appeared that the reinforcement-soil resistance is a important factor rather than the stiffness of the reinforcement.

Giroud and Noiroy (1981) have made a quasi-static analysis of the unpaved road behaviour with and without geotextile as a reinforcement. They considered the membrane effect of the geotextile which contributes to an increase in the bearing capacity of unpaved road by placing the geotextile at the interface between the sub base and aggregate layer.

Bourdeau et al.(1982) proposed a theoretical model for soil-membrane interaction. They analyzed the membrane action in the geotextile in improving the load-settlement response of a two layered soil.

Ingold and Miller (1982) proposed the concept of a composite material theory in which the confining effect of the reinforcement is assumed to impart an equivalent shear strength (Cu) to the soil. Based on this concept, they derived an equation for the bearing capacity of reinforced cohesive soil and compared it with the model test results which showed a close agreement.

Bouer and Preissner (1986) conducted model tests to determine the effect of geotextiles on bearing capacity and settlement behaviour of a two layer soil system. Then they analyzed the same problem by a combined approach which considered membrane effect and the shear resistance of the reinforcement separately .

Madhav and Ghosh (1990) presented a three parameter foundation mudel to quantify the effect of the tensile force on settlement and bearing capacity of granular fill underlain by a soft soil with the geotextile placed at the interface. Their result showed an unreinforced granular fill over soft soil had a

significant influence in improving the load response and the introduction of the geotextile at the interface improves the same further, the improvement was found to be is more in case the soil was softer.

Florkiewicz (1990) analyzed the problem of bearing capacity of soil with a reinforcement layer based on the assumption of classical concept of reinforced earth (rigid perfectly plastic Coulomb's model with anisotropic pseudo-cohesion) Both the soil and the reinforcement become plastic at the same time. He compared his results with the experimental data of Binquet and Lee (1975) and showed a good agreement with his results.

Houlsby and Jewell (1990) proposed a design method for reinforced unpaved roads for small rot depths. The analysis was based on the concept that the principal function of the reinforcement is to carry shear stress, which are caused by the vertical loading. The magnitude of the shear stress is calculated from the assumption that the fill beneath the footing is in a state of active failure and that the fill on each side of the footing is in a general state of passive failure.

Based on combined membrane action and lateral restraint Sellmeijer (1990) presented a new model for predicting the behaviour of soil-geotextile-aggregate system,The behaviour of the individual components viz. aggregate, geotextile and soil were modeled by elasto-plastic shear theory, the membrane action and the bearing capacity respectively. The pre[illegible] models predict large displacements in order to assure stat[illegible] derived from the membrane effect. But this paper incorporat[illegible] ffect of lateral

restraint which assures a slab effect of the aggregate and which reduces the deformation greatly. This method thus can be adopted for design of roads in cases where the deformation is less .

CHAPTER III

ANALYSIS OF FOUNDATIONS ON REINFORCED BEDS

3.1 INTRODUCTION

Large number of experimental results are available based on model and field plate load tests over reinforced homogeneous sand beds and reinforced granular materials overlying soft soils. The available data is analysed to find general trends and to determine the effects of various parameters on the bearing capacity and settlement behaviour of reinforced soil beds. The test details and the results are described briefly as follows and is summarised in Tables 3.1 and 3.2.

3.2 DESCRIPTION OF THE MODEL TESTS

Binquet and Lee (1975) conducted model tests with a 76mm strip footing on reinforced sand foundation for three conditions (i)homogeneous deep sand, (ii) sand above an extensive layer of very soft material simulating soft clay or peat and (iii) sand above a finite size pocket of very soft material such as a pocket of organic soil or a cavern in lime stone. The soil used for model test was Ottawa sand with D_{50} =0.75mm, Cu=1.5, γ_{dmin}=13KN/m^3 and γ_{dmax} =16.4KN/m^3. The reinforcing element was 13mm wide aluminum strips, whose breaking strength was 17N. The model tests were conducted in a sand box 1.5m long, 0.51m wide and 0.33m deep.

Akinmusuru and Akinbolade (1981) carried out model tests with a square footing on a sand bed reinforced by flat strips of rope fibre, whose width was 10mm and strength 80N/mm^2. The soil

used was uniformly graded dry sand with D_{50}=0.43mm, G=2.7, γ_d=17KN/m^3 and friction angle of 38^o. They also conducted a series of tests by placing layers of crushed rock in between the sand beds and compared the results with the same obtained from the previous series of tests using geofabric as reinforcement.

Saran and Talwar (1981) conducted model strip footing tests to investigate the influence of the depth to the first layer of reinforcement and length of the reinforcing strips on the bearing capacity of the reinforced beds. The tests were performed in a wooden tank 700mm long, 503mm wide and 580mm deep, stiffened by angle irons. The model strip footing 500mm long, 50mm wide and 25mm thick had a rough base. The properties of the sand used were D_{50}=0.13mm, Cu=1.85, γ_d=16.5 KN/m^3 and Id=79.5%. The reinforcing element was fibre glass strip 160mm wide and having a tensile strength of 13.48 KN per cm width.

Patel et al. (1981,1982) studied the effect of the shape of footing on the bearing capacity of reinforced sand bed. The footings were prepared from 20mm thick mild steel plate having the following dimensions: circular footing-145mm diameter, strip footing-145mm wide and rectangular footing 113.5mm×465mm. The circular and the rectangular footings were tested in a big tank 1080mm×930mm in plan and 730mm high. The strip model footings were placed across a narrow tank of 340mm×1000mm in plan and 700mm in depth. *Ennore* sand and fine *Baderpur* sand with the angles of shearing resistance of 37^o and 42^o respectively were used for preparing the lower virgin part of sand bed. The sand used in forming the thin layer of reinforcing element was medium *Baderpur*

Tab. 3.1. DETAILS OF THE TEST RESULTS ON FOUNDATION BEDS WITH HOMOGENEOUS SAND AND HORIZONTAL REINFORCEMENT

Foundation Type	Soil Type	Reinforcement Type	Quantity u/B	1/B, z/B	z/B	N	Strength/ CR	Improvement BCR	MR	Reference
Strip	Sand	Aluminium	0.330	20.000	0.330	4.000		2.400		BInqeet & Lee(1975)
	D =0.75mm	Strips	0.670					1.900		"Bearing Capacity Tests
	C =1.5	13mm wide	1.000	(variaion of u/B)				1.600		Reinforced Earth Slabs
	γ_d=15KN/m³	width=35								
	φ=35°	t=0.013mm		20.000	0.330	1.000		1.050		
		S=17N				3.000		1.420		
				(variaion of N)		4.000		2.240		
						5.000		2.620		
						6.000		3.100		
Square	Sand	Rope Fibre	0.5	10.000	0.500	5.000		2.050		Akinmusuru & Akinbolade
	D =0.43mm	S =80N			0.750			1.850		
	D =0.14mm		(variation of z/B		1.000			1.600		"Stability of Loaded
	G=2.7				1.500			1.400		Footings on
	γ_d=17KN/m³									Reinforced Soil"
	φ=38°		0.250	10.000	0.500	5.000		1.850		
			0.500					2.050		
			0.750	(variation of u/B)				1.550		
			1.000					1.300		
			1.500					1.250		
			0.750	10.000	0.500	1.000		1.180		
						2.000		1.560		
						3.000		1.500		
				(variation of N)		4.000		1.530		
						5.000		1.530		
Strip	Sand	Fibre Glass	0.250	8.000	0.500	8.000		4.690	1.490	Saran & Talwar(1981)
	D =0.1	Cloth Strips	0.500					3.690	1.490	"Laboratory Investigation
	C =1.8	Width=15mm	1.000	(variation of u/B)				2.350	1.300	of Reinforced Earth
	e =0.927	Tensile St.=13.5KN/m								Slabs"
	γ_d=16KN/m³	ii.Aluminium Foil	0.250	14.000	0.500	8.000		2.620	1.520	
		Tensile St.=.33KN/	0.500					2.580	1.210	
			0.750					2.270	1.090	
			1.000	(variation of u/B)				1.770	1.090	
			1.500					1.150	1.000	
			0.500	1.000	0.500	8.000		2.230		
				2.000				3.230		
				3.000				3.350		
				4.000	(variation of 1/B)			3.540		
				5.000				3.690		
Strip	Sand	Geotextile	0.125	16.700		1.000		1.340		Andrews et al. (1983)
	D =0.85mm	weight=140gm/m	0.250					1.330		"The Behaviour of a
	Cu=1.22	Thickness=0.7mm	0.500	(variation of u/B)				1.240		Geotextile Reinforced
	φ=42.7°	Strength=6.7KN/m	0.750					1.090		Loaded by a Strip"
								3.000		
							Dr			
Rectangle	Sand	Aluminium Foil	0.330	8.000	0.330	3.000	51.000	1.600	1.420	Richard J.Fragaszy(1984
	Cu=1.8						61.000	1.700	1.310	"Bearing capacity of

	$\gamma_{d\,max}$=16.1KN/m^3		(variation of Dr)				80.000	1.700	2.210	Subgrades"
	$\gamma_{d\,min}$=13.7KN/m^3						90.000	1.600	1.760	
			0.330	3.000	0.330	3.000	70.000	1.260		
				4.000				1.320		
				5.000				1.550		
				6.000	(variation of 1/B)			1.640		
				7.000				1.650		
				8.000				1.670		
Square	Sand	Geotextile	(E calculated at s/B=0.025			1.000			1.110	Guido et al.(1985)
	Cu=2.5		ie. at 1/2 of of ultimate			2.000			1.220	"Bearing capacity of a
	D =0.086m		in case of unreinforced			3.000			1.360	Geotextile Reinforced
	D =0.18mm		section)			4.000			1.360	Foundation"
	$\gamma_{d\,max}$=16.5KN/m^3									
	$\gamma_{d\,min}$=13.45KN/m^3		0.500	2.000	0.250	1.000		1.350		
	Dr=50%					2.000		1.500		
				(variation of N)		3.000		1.800		
						4.000		1.850		
			0.500	1.000	0.250	2.000		1.220		
				1.500				1.300		
				2.000				1.560		
				2.500	(variation of 1/B)			1.500		
				3.000				1.610		
Square	Sand	Geogrid	0.250	3.000	0.250	3.000		2.560		Guido et al.(1987)
	Cu=1.9	Tensar SS1	0.500					1.750		"Comparision of Geogrid
	Cc=1.23		0.750					1.600		and Geotextile Reinfor
	D =1.23mm		1.000	(variation of u/B)				1.250		Earth Slabs"
	D =.086mm		1.250					1.200		
	$\gamma_{d\,max}$=13.1KN/m^3									
	$\gamma_{d\,min}$=15.6KN/m^3		0.500	3.000	0.125	2.000		2.840		
	φ =37°				0.250			2.530		
					0.500			2.160		
			(variation of z/B		0.750			1.840		
					1.000			1.500		
					1.250			1.440		
			0.500	3.000	0.250	1.000		1.420		
						2.000		1.670		
			(variation of N)			3.000		1.720		
						4.000		1.720		
			0.500	1.000	0.250	3.000		1.370		
				1.500				1.650		
				2.000				1.650		
				2.500	(variation of z/B)			1.670		
				3.000				1.750		
		Geotextile	0.500	3.000	0.250	1.000		1.250		
						2.000		1.370		
			(variation of N)			3.000		1.700		
						4.000		1.630		
			0.500	1.000	0.250	3.000		1.210		
				1.500				1.370		

				2.000				1.500		
				2.500	(variation of 1/b)			1.540		
				3.000				1.670		
Strip	Sand	Aluminium	0.300	10.000		3.000		2.000		Sreekantiah H. R.(1988)
	G=2.52	Foil	0.400					1.860		
	e =0.958	Strength=17.36KN/m	0.500					1.770		"Stability of Loaded
	e =0.711		0.600	(variation of u/B)				1.730		Footings on
	Dr=65%	(Grid Type)	0.700					1.680		Reinforced Sand"
	γ_d =15KN/m^3									
			0.300	10.000		2.000		1.740		
						3.000		2.000		
						4.000		2.030		
				(variation of N)		5.000		2.440		
						6.000		2.810		
						7.000		2.960		
Square	Sand	G.I.Strip	0.240	5.000		1.000		1.630		Sankariah & Narahari(19
	e =0.36		0.470					1.500		"Bearing Capacity of
	e =0.62		0.710					1.540		Reinforced Sand
	Dr=72.8%		0.940	(variaton of u/B)				1.420		Beds"
	γ_d =18KN/m^3		1.410					1.410		
	φ =48°									
			0.250	5.000	0.240	1.000		1.630		
						2.000		1.750		
				(variation of N)		3.000		2.050		
						4.000		2.500		
		Welded Mesh	0.240	5.000	0.240	1.000		1.900		
						2.000		1.920		
				(variation of N)		3.000		3.360		
						4.000		4.060		
Strip	Sand	Mild Steel	0.300	1.000	0.300	1.000		1.320	1.070	Chung & Tatsuoka(1990)
	(Toyura)	Bar				2.000		1.670	1.170	"Bearing Capacity of
	Dr=80-86%			(variation of N)		3.000		2.100	1.340	Reinforced Horizontal
	D =0.16mm					5.000		2.480	1.370	Sandy Ground"
	Cu=1.46									
	G=2.64									
	γ_{dmax}=16.11KN/m^3			1.000		3.000		2.100		
	γ_{dmin}=13.01KN/m^3			2.000				2.710		
				3.500	(variation of u/B)			2.470		-
				6.000				2.810		
				6.000		1.000		1.420	-	
				(variation of N)		2.000		2.150	1.000	
						3.000		3.220	1.230	
							CR			
				2.000		3.000	0.045	1.620	1.000	
							0.090	2.180	1.110	-
							0.180	2.710	1.230	
							(Er Ar) 10N			
				2.000		3.000	18.300	2.180		
							18.300	2.020		
				(variation of strength)			3.660	2.170		
							1.050	1.960		

Table 3.2 DETAILS OF THE TEST RESULTS ON REINFORCED FOUNDATION BEDS WITH GRANULAR LAYER OVER SOFT SOIL AND HORIZANTAL REINFORCEMENT.

Foundatio Type	Soil Type	Reinforce ment Type	u/B	1/B,b/B	QUANTY Position	N	H/D,H/B	BCR	MR at 10% s/B	Remark	Reference
Circle	Granular Fill	Steel Bars		Sand		0.000	0.250		1.510		Milovic, M.C.(1984)
	over Soft Soil	Dia.=12mm					0.500		1.890		"Field Load Tests on
	i.Gravel over Sand						0.670		2.300		the Reinforced Two Lay
	ii.Gravel over Soft Soil										System"
						1.000	0.250		2.210	1.460	
	Sand					2.000	0.250		3.000	1.970	(Improvement Due to
	γ_d=15+.5KN/m^3					3.000	0.500		4.730		the Reinforcement)
	Cu=1.5										
	Soft Soil			Soft Soil		0.000	0.250		1.230		
	C=15KN/m^2						0.500		1.380		
	φ=13°						0.670		1.650		
	W_n=28.3%										
	W_L=27.15%					1.000	0.250		1.920	1.580	(Improvement Due to
						2.000	0.250		2.480	2.040	the Reinforcement)
						3.000	0.500				
Strip	Sand over	Geotextile		13.250		0.000	1.000		1.540		
	Soft Soil						0.500		1.280		
	Sand	t S in KN/5cm width									
	γ=16.72Kn/m^3	3 243					1.000		1.420	at 25%	
	φ=30°	5 325					0.500		1.420	s/B	
	Dr=69%										
	Mud			5.000			1.000		1.380		
	W_n=16.5%						0.500		1.080		
	φ=9°										
	W_L=54%			13.250			0.500		1.310		
				5.000			0.500		1.090		
Strip	clay subgeade	Geogrid		16.300			1.270		1.000		Milligan et al. (1986)
	Wp=24%	Tensar SS2					0.670		1.170		Model and Full Scale
	Wl=78%										Tests of Granular Layers
Circle	Granular fill						1.230		1.000		Reinforced with a
	well graded crushed lime stone						0.920		1.230		Geogrid"
							0.670		1.230		
Strip	Subgrade	Geogrid		18.200			1.500		2.830		Jarret,P.M.(1986)"Load Test
	PEAT	Tensar SS2					0.750		2.000		onGeogrid Reinforced Gravel
	Wl=85%										Fills on Peat Subgrade"
	Cu=4KPa			18.200			1.500		2.330	1.170	(Improvement Due to
							0.750		3.330	1.170	the Reinforcement)
Circle	Subgrade	Geotextile		12.700			1.000			1.700	Resl and Warner(1986)"The
	i.loose sand,CBR<3	Wt=280gm/m					1.670			1.290	Influence of Nnwoven
	ii.lacustrine clay CBR<4.5%	t=2.6mm									Needle Punched Geotextile
		St=800N/5cm		12.700			1.000			1.430	on the Ultimate Bearing
	Fill Material						0.670			1.320	Capacity of Subgrade"
	i.sand										
	ii.sandy gravel										
Strip	Granular Fill	Geogrid						Cu=6Kpa	Cu=9Kpa	Cu=14Kpa	Love et al.(1987)"Analytica

	over Clay	Tensar SS1				0.670	1.330	1.040	1.000	and Model Studies of
	(Kaolin)					1.000	1.660	1.110	1.000	Reinforcement of Layer of
						1.330	1.830	1.300	1.140	Granular Fill on a Soft
										Clay Subgrade"
						0.670	1.000	1.430	1.500	(Improvement Due to
						1.000	1.080	1.500	1.240	the Reinforcement)
						1.330	1.220	1.400	1.420	
Strip	Granular Fill	Geotextile				0.250		1.050		Dembicki et al.(1988)
	Fine Sand					0.500		1.210		Influence of Geotextile
	Cu=1.6					0.750		1.680		on Bearing Capacity of
	γ_d=16.9KN/m^3					1.000		1.950		Two Layer Sub-soil"
	ϕ=30					1.500		2.420		
	Clay Subgrade									
	γ_d=20.4KN/m^3					0.250		1.150	1.050	(Improvement Due to
	Wn=20.5%					0.500		1.260	1.050	the Reinforcement)
	Wl=27%					0.750		1.560	1.060	
	Cu=11Kpa					1.000		1.910	1.100	
						1.500		2.500	1.100	
Circle	Sand Fill	Geotextile				0.500		1.620		Sridharan et al.(1988)
	γ_d=16.2KN/m^3					1.000		1.370		"Reinforced Soil
	Dr=65%					1.500		1.640		Foundation
	ϕ =41°			position	N					on Soft Soil"
	Cu=2				0.000	0.500		1.310	1.000	
	D =			.25D	1.000	0.500		1.420	1.460	
	Subgrade			.16D,.33	2.000	0.500		1.550	1.620	
	Saw Dust			.25d,.4D	2.000	0.500		1.590	1.920	
	γ_d=.39KN/m^3									
	ϕ=13°									
Strip	Sand Fill	Geotextile	10.000			0.250	1.090			
	γ_d=17.01KN/m^3	St=534N				0.500	1.330			Das.M(1988)"Shallow
	ϕ=43.5°					1.000	2.070			Foundation on Sand
	Clay Subgrade					1.500	2.730			Underlain Underlain by
	Wl=34%					2.000	2.860			Soft Clay with
	Wp=21%					2.500	2.860			Geotextile Interface"
	γ_d=20.12KN/m^3					3.000	2.860			
	Cu=14KPa									
			10.000			0.250	1.350			
						0.500	2.040			
						0.750	2.980			
						1.000	2.940			
						1.250	2.800			

sand whose angle of friction was 44.5^{o}. The reinforcement used was fibre glass woven roving, weighing 360 gm. per metre sq. and of high tensile strength of 3.6 GN/m^2.

Andraws et al. (1983) studied the effect on bearing capacity of a strip footing resting on a bed of dense sand reinforced by a single layer of geotextile at different depths. The model tests were conducted in a box of size 2.0m x 0.3m x 1.14m filled with sand and the strip footing was a 120mm wide rigid, smooth steel plate. The sand had a particle size of D_{50}=0.85mm, uniformity coefficient, C_u= 1.22 and angle of internal friction, ϕ = 42.7^{o}. The geotextile used had a mass per unit area of 140gm/m^2, average thickness of 0.7mm and strength of 6.7 KN/m at 40% strain.

Milovic (1984) conducted two series of field load tests with a rigid circular foundation of diameter 60cm on a reinforced soil. In the first group of tests the upper gravel layer was reinforced by steel bars and the lower layer was a natural soil deposit, with a dry density of 15KN/m^3 and C_u= 1.5. The modulus of deformation determined by field load tests was $E_s \simeq$ 11 MN/m^2. In the second group of field load tests the lower layer was a soft silty clay whose properties were W_n= 28.3%, W_L= 27.15%, ϕ = 13^{o}, c=15KN/m^2 and compressibility modulus E_{oc}= 2700KN/m^2. The tests were conducted varying the thickness of the upper gravel layer reinforced by more than one layer of reinforcing elements made of steel bars of diameter 12mm. The effects of both the thickness of the gravel layer and the number of reinforcing layer on the bearing capacity of the footing were studied.

Fragaszy and Lawton(1984) studied the influence of soil density and the length of reinforcing strip on the bearing capacity of reinforced sand. A rectangular steel footing composed of three segments 75mm wide and 152mm long were used for the laboratory experiments. The tests were conducted in a box measuring 1.22m long, 0.56m wide and 0.36m deep. The sand used was and angular, uniformly graded sand, with D_{50}= 0.4mm and C_u= 1.8. The friction angles were 36.5^o, 38^o and 39^o at D_r= 31%, 70% and 90% respectively. The reinforcing strips were cut from household aluminium foil 25.4mm wide and 0.0254mm thick.

Guido et al. (1985,87) carried out model tests with geotextiles and geogrids as the reinforcing elements. They studied the effects of the tensile strength of the geotextile apart from the other parameters on the bearing capacity of the reinforced soil. They used a 0.31m wide square footing resting on a sand fill box made of square, stiffened plexiglass, the size of which was 1.22m x 1.22m x 0.92m(deep). Two series of tests were conducted on sands of different properties. The sand used in the first series had D_{50}= 0.18mm, C_u= 2.5, γ_d= 14.8KN/m^3, D_r= 50% and ϕ = 35^o. In the second series of tests the above soil parameters were chosen as D_{50}= 0.15mm, C_u= 1.9, γ_d= 14.39KN/m^3 and ϕ = 36^o. The strength and other properties of geotextiles and geogrids were as follows.

Structure	Thickness $\times 10^{-4}$ mm	Tensile Strength KN/m
Nonwoven	3.8	0.67
Nonwoven	27.9	1.16
Woven	7.6	1.33
Nonwoven staple	22.9	1.47
Woven	7.6	1.78
Nonwoven	53.3	2.16
Tensar SS1	-	12.50
Tensar SS2	-	17.90
Tensar SS3	-	16.00

Dembicki and Alenowicz(1985,87) conducted model tests on two layer subsoil consisting of a layer of sand fill on the surface of soft clay. The box used for the experiments had a length of 2.65m, width of 0.5m and height 1.07m. The plate used as a strip footing was 0.2m wide. The properties of sand fill were D_{10}= 0.18mm, γ_d= 16.72KN/m^3, ϕ = 30^o, D_r= 69% and those of the underlaying mud layer were γ_d= 20.9KN/m^3, W_L= 16.5% , C = 5.5KPa and I_L= 0.59. Two types of geotextiles having different strengths viz. 4.8 and 6.9KN/m, were used and were placed at the interface between the two layers.

Two strip footings placed at a spacing of 60cm c/c, to simulate the loading from a truck axle were studied. In both the cases the effects of five different thickness of fill layer with and without geotextile layer at the interface were investigated.

Love et al. (1984,87) carried out model tests to determine the influence of a geogrid layer placed at the base of the granular fill on the surface of soft clay. The clay was kaolin, consolidated from a slurry and then allowed to swell to produce a fully saturated over consolidated clay subgrade. The vane shear strength at a depth of 75mm were 6, 9 and 14KPa in the three series of tests. The width of the strip footing was 75mm. The measurements of the load on the footing were made by transducers and the deformation of the soil were measured photographically.

Ingold and Miller(1982) carried out model tests to determine the increase in bearing capacity of a cohesive soil by reinforcing it with a geogrid. The test box was 710mm long, 150mm wide and 150mm deep. The width of the strip footing was 50mm. London clay was the soil used for the experiments.

Milligan et al. (1986) conducted large scale tests with strips and circular footings resting on a two layer subsoil. The strip footing was 300m wide and 1500mm long representing plane strain condition and the diameter of the circular footing was 300mm. The tests were conducted on a test bay 16.8m long, 4.8m wide and 0.96m deep, with vertical concrete side walls. Partition boards were used to isolate the total pit into eight sections. The subgrade material was a heavy clay with plastic and liquid limit of 24 and 78% respectively. The geogrid reinforcement was Tensar SS2 whose strengths were 18 and 32KN/m in the longitudinal and transverse directions with strain at peak load of 12%. The geogrid was placed at the interface between the subgrade and the granular fill. The granular fill was made of well graded crushed limestone.

Jarret(1986) carried out large scale plain-strain loading tests in the laboratory on a series of thin gravel fills compacted over a peat subgrade. The tests were carried out in test pit 3.7m × 2.4m in plan and 2.0m deep. The peat subgrade was made by consolidating Sphagnum peat at an average water content of 85% up to a depth of 0.9m and with an average vane shear strength of 4.0KPa. Gravel fills were compacted on this subgrade using a well graded 20mm crushed lime stone aggregate. The model strip footing consisted of a beam of width 203mm spanning full 2.4m width of the pit. The thickness of the gravel fills were 150, 300 and 450mm with Tensar SS2 geogrid placed at the interface.

Resl and Warner(1986) conducted plate load test on compacted granular material underlain by a soft soil, with and without placing the geotextile at the interface. The test was carried out in a test pit of 2.8m × 3.8m in plan with 0.8m deep fill of two types of soil. The sub-base course consisted of (i)loose sand (CBR <3%) and (ii)medium plastic silt (CBR <0.5%). The top fill layers used were (i)compacted sand and (ii)sandy gravel. The geotextile was Polyfelt-Ts-700 whose unit area weight = 280gm/m^2, thickness = 2.6mm and tensile strength = 16KN/m. The load was applied through a 300mm diameter rigid plate.

Jarret and Bathurast(1986) conducted large scale model test to compare load deformation performance of a gravel in-filled geoweb/geocell mattresses and unreinforced gravel bases over a peat subgrade under plane strain loading. The geoweb reinforcement was produced by welding nonperforated plastic strips ultrasonically. The geocell reinforcement was constructed from strips of polymeric

mess(geogrids) attached by metal bodkins. The tests were performed in a test pit measuring 2.4m wide, 3.6m long and 1.8m deep. The vane shear strength of the peat subgrade before placing the fill was 3±1KPa. The granular fill was of good quality crushed limestone aggregate of maximum size 20mm. The average density of the compacted gravel layer was 1950 ± 50Kg/m^3.

Gardel and Morel(1986) experimented with various types of geotextiles elements to increase the bearing capacity of the soil. *Texol* is a three dimensional fibre-soil composite is obtained by mixing continuous textile threads with the soil. *Armater* is a regular and continuous honeycomb structure manufactured by bonding geotextile strips with the bottom-less cell having hexagonal shape. *Nidapalast* also has a regular honeycomb structure constituted from elementary polypropylene meshes. For the test programme a test bed of 2m × 2m in plan and 1.4m deep, with rigid metallic side plates was constructed. The subgrade soil was silt with W_L=34%, I_P= 10% and γ_d= 17.5KN/m^3.The footing used was a circular plate with 300mm diameter. The sand used for preparation of *Texol* was of γ_d= 15.7KN/m^3 ϕ = 39^{o}and that used in case of *Armater* and *Nidaplast* had mean unit weight of 14.3KN/m^3.

Verma and Char(1986) carried out plane strain model test with a 100mm wide model footing to study the influence of vertical reinforced sand bed. The tests were carried out in 720×400×90mm box. The uniformity coefficient and the effective size of the sand used were 1.41 and 0.49mm respectively and the dry density of the sand was 15.8KN/m^3 at relative density of 71%. Galvanised iron rods were used as reinforcement and were pushed at specified spacing

after preparing the sand bed.

Sridharan et al. (1988) conducted model tests on a two layer soil system in a tank of size 750x750x600mm. A circular rigid mild steel plate of 50mm thick and 150mm diameter was used as model footing. The soft soil was simulated by saw dust which had very low shear strength and high compressibility. An uniformly graded sand was used as the fill material above it. The properties of the sand were D_{50}= 0.95mm, C_u= 2.0mm, γ_d= 16.2KN/m^3, D_r= 65% and ϕ =41^o. The reinforcing element was geogrid and geotextile placed at different depths. The effects of thickness of fill material, position of reinforcement and number of reinforcing layers on the bearing capacity of the footing was studied.

Sankariah and Narahari(1988) studied the effect of three types of reinforcing elements on the bearing capacity of reinforced sand bed. The tests were conducted in a wooden tank 450mm square in plan and 280mm in deep. The square footing made of wooden plank was 85mmx85mm. The sand had e_{max}= 0.62, e_{min}= 0.36 and ϕ = 40^o at D_r=72%. The dimensions and breaking strengths of the reinforcing elements were as follows

	Width (mm)	Thickness (mm)	Breaking strength KN/m^2×10^3
GI strips	5.0	0.15	50.0
Bamboo strips	10.0	1.00	1.7
Mildsteel welded mesh	1.0	1.00	6.0

Das(1988,89) conducted model tests with square and strip shallow foundations on two layer soil. The tests on the strip

foundations were conducted in a box measuring 915mm×304.8mm in plan and 762mm deep. The size of the box for the square foundation was 915mm×915mm×762mm. The foundation used had the dimensions of 76.2×308.4mm for strip footing and 76.2×76.2mm for the square footing. The clay used for the lower layer had W_L= 38%, I_P= 21%, C_u= 14KPa and γ_d= 20.12KN/m^3. The upper sand layer had γ_d= 17.01KN/m^3 and ϕ = 43.5^o. The geotextile which was used at the interface of the two layers of soil had a tensile strength of 534N at elongation of 50%.

Samtani and Sonpal (1988) conducted model strip footing tests on cohesive soil reinforced with metallic strips. A square tank 914.5mm×914.5mm in plan and 762mm deep made of thick wooden panels was used for the model study. The footing was made of stiff cast iron strip of size 495.3mm×76.2mm. The soil used was typical black cotton soil with W_L= 55.5%, I_P= 27% and C_u= 15.94KPa. The reinforcing strips were cut from a 0.55mm thick aluminium foil to a width of 20mm. The tensile strength of these strips were 1.08N/mm^2.

Chung and Tatsuoka (1990) carried out a series of plane strain model test on horizontally reinforced sand bed, with a 100mm wide model strip footing. The homogeneous sand layers were made by pouring them from a fixed height which gave a relative density of 80-86%. The sand used was *Toyura* sand having D_{50}=0.16mm, C_u=1.46, G=2.64, γ_{dmi}=13.09KN.m^3 and γ_{dmax}=16.11KN/m^3. Phosphor bronze and aluminium foil were used for reinforcement whose elastic modulii were 1.22×10^{8}and 0.7×10^{8}KN/m^2. The various parameters studied were the effect of length of reinforcement, the number of reinforcement layers, the covering ratio, the effect of rigidity and rupture

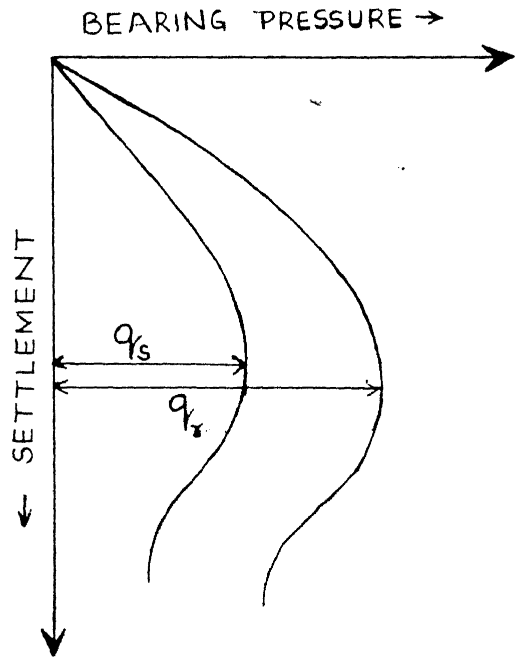
strength of reinforcing element.

Puri and Das(1990) conducted model tests to study the effect of vertical reinforcement on improving the bearing capacity of a strip footing supported on a sand layer reinforced by semi flexible thin metal rods. The tests were conducted in a box measuring 1140x101.6x915mm with strip footing of 304.8mm wide. Copper rods having diameter 2mm were used as the vertical reinforcement. The tests were carried out placing the soil at two dry densities of 15.61 and 16.04KN/m^3 and the friction angles equal to 36.2^{o}and 40.3^{o}respectively.

Shimizu and Iui(1990) carried out model tests on a sand bed reinforced with a six sided geotextile cell, with the loads applied at the center of the cell. The size of the cell, the material of geotextile and the loading area were varied to find the optimal conditions for improving the bearing capacity.

3.3 ANALYSIS OF TEST RESULTS

The model tests as reported earlier were conducted by using different type of reinforcing material like aluminium foil, mild steel rods, geotextiles and geogrids, and using the locally amiable granular and soft soils. Thus there was a wide variation in the results depending on the test conditions and the testing material used. In this study it is proposed to identify the general trends in the increase of bearing capacity by reinforcing the soil mass. Hence all the available results were analysed to find the nature of improvement in terms of Bearing Capacity Ratio(BCR) and Modular Ratio(MR) as defined in the following figure.



$$BCR = \frac{q_r}{q_s}$$

where q_r and q_s are the ultimate bearing capacities of the reinforced soil bed and the soil alone

$$MR = \frac{E_r}{E}$$

where E_r and E_s are the deformation modulus of the reinforced soil and unreinforced soil respectively

3.3.1 HOMOGENEOUS SAND AND HORIZONTAL REINFORCEMENT

When a bed of homogeneous sand mass is reinforced by providing a reinforcing material, the improvement in the bearing capacity and the settlement behaviour depend on a number of parameters(Table.3.1) viz. number of reinforcement layers, depth to the first layer of reinforcement, length of the reinforcement, vertical spacing of the reinforcement and strength of the reinforcement. The effect of all these parameters have been studied and described as follows.

(i)Effect of Number of Reinforcement Layer

With the increase in the number of reinforcement layers, the bearing capacity as well as the settlement behaviour improves as shown in the Figs.3.1 to 3.3. Fig.3.1 shows that for strip footings the bearing capacity ratio(BCR) increases with the increase of the number of reinforcing layers. The general trend of increase of BCR with the number of reinforcing layers(N) is linear upto N value equal to 6. Beyond this value of N data presented by Sreekantiah (1988) shows a tendency for the BCR to reach a limiting value of about 2.5 at N equal to 7. In case of Chung and Tatsuoka(1990), who had reported a much improved behaviour of the foundation with only three layers of reinforcement, the increase in BCR may be attributed to the reinforcement material(mild steel) they have used whose rapture strength is much greater than the aluminum foil or geotextiles and geogrids.

Fig.3.2 shows the effect of the number of reinforcing layers on BCR for square footings. It can be seen from the general trend of the data that as N increases BCR also increases initially at an increasing rate, but beyond a N value of 3 it reaches a limiting value of about 1.5. This is in contrast to the behaviour as shown in Fig.3.1 for strip footing where the limiting value did not reach up to N value equal to 6.

Fig.3.3 shows the influence of the number of layers on the modular ratio MR. It can be observed that as the number of the reinforcing layers increase MR also increases within the studied parametric range. The variation of MR with N is nonlinear.

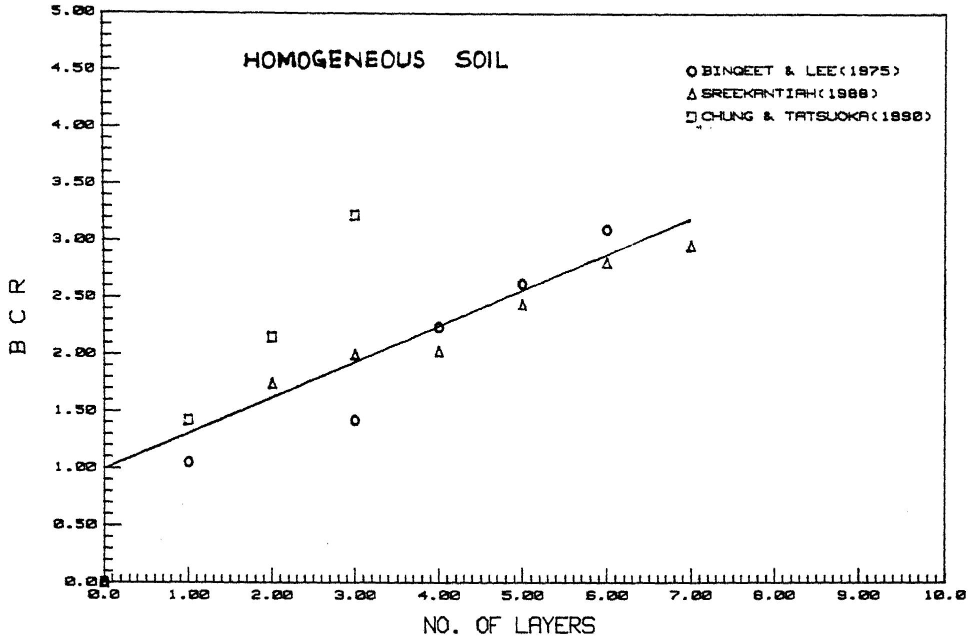


Fig.3.1 EFFECT OF NUMBER OF LAYERS (N) ON BCR
(STRIP FOOTING)

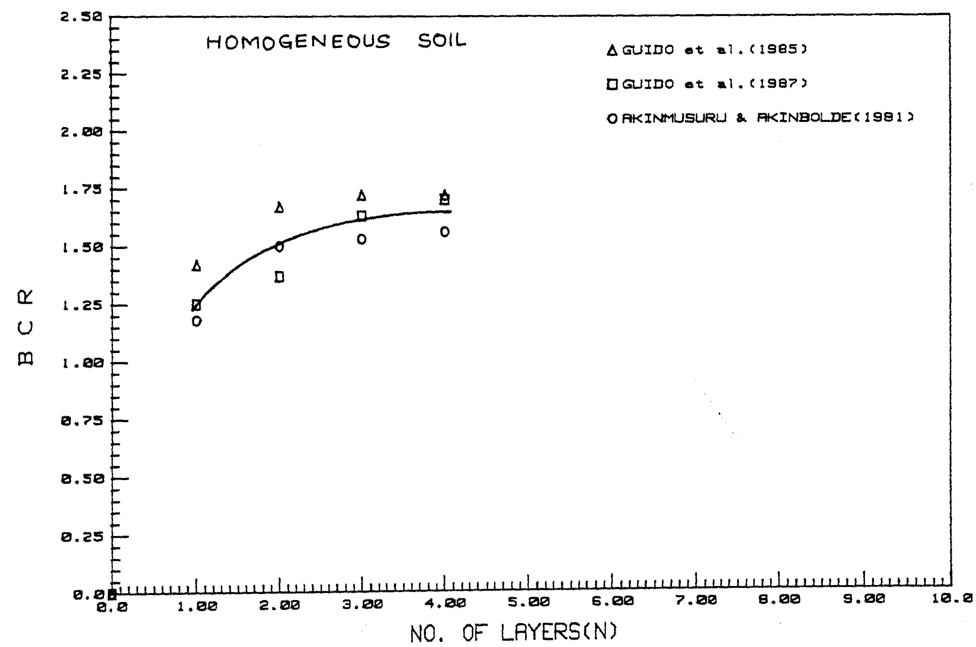


Fig.3.2 EFFECT OF NUMBER OF LAYERS (N) ON BCR (SQUARE FOOTING)

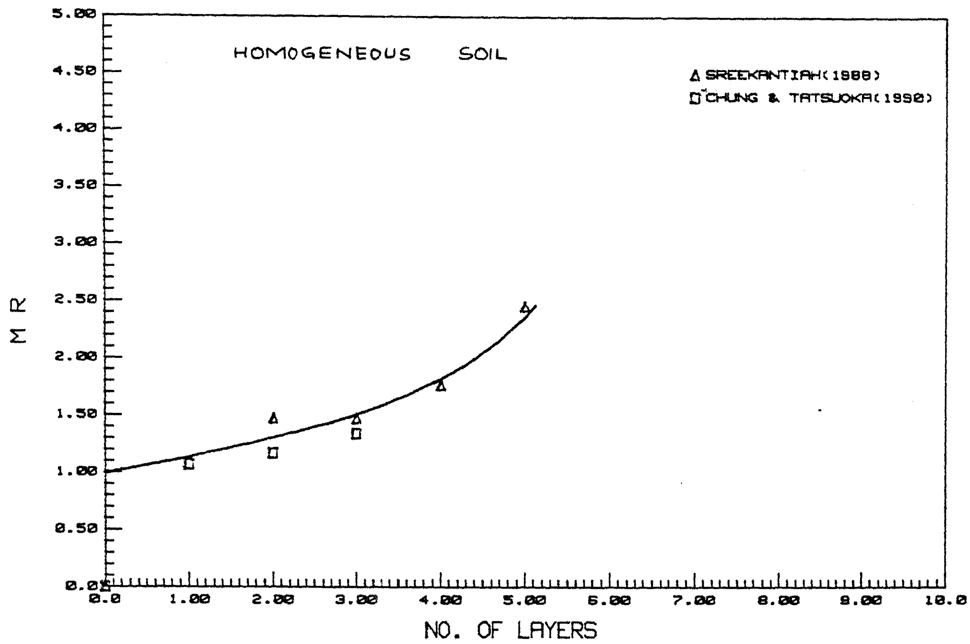


Fig. 3. 3 EFFECT OF NUMBER OF LAYERS(N) ON MR (STRIP FOOTING)

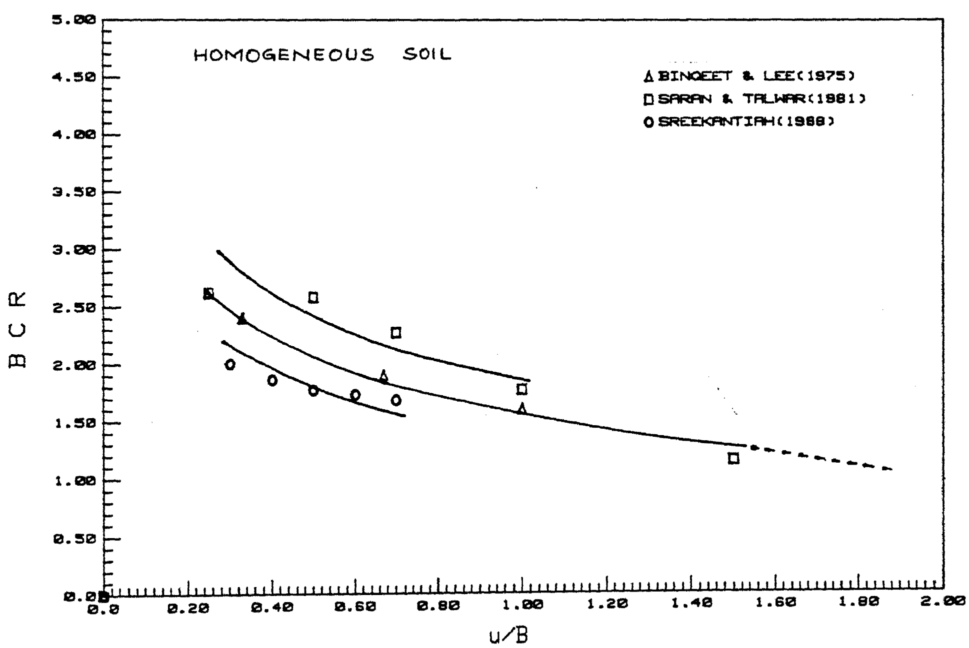


Fig. 3. 4 EFFECT OF u/B RATIO ON BCR (STRIP FOOTING)

(ii) Effect of Depth to the First Layer of Reinforcement

The effect of u/B ratio, ie.the depth to the first layer of reinforcement on the bearing capacity ratio of the foundation is shown in the Figs.3.4 and 3.5. It can be observed from these figures that with the increase in the u/B ratio BCR gradually decreases and tends to a value of 1.0 for u/B equal to 1.0.(Fig.3.4). This may be due to the fact that when the first layer of reinforcement lies at a depth greater than 1.5B, the foundation fails above the reinforcing zone and the reinforcement has no effect on the bearing capacity.

(iii) Effect of the Length of the Reinforcement

Fig 3.6 shows the effect of the length of the reinforcement denoted by l/B ratio on BCR for strip footings. It can be observed from this figure that with the increase in l/B ratio the BCR increases upto around 3 to 3.5 after which the increase is minimal. Beyond l/B ratio equal to 6 the increase in BCR is very small. As the length of the reinforcement is increased, it provides more anchorage, thus increases the bearing capacity.Chung and Tatsuoka(1990) have reported that even by providing the length of reinforcement, equal to the width of the footing, the bearing capacity can be increased appreciably. They attribute this to the changed pattern of strain developed by the inclusion of reinforcement in the soil mass.

Fig.3.7 shows the effect of l/B ratio on BCR for square footings. In this case also with the increase in l/B ratio BCR increases, but beyond l/B equal to 3 the change in BCR is not

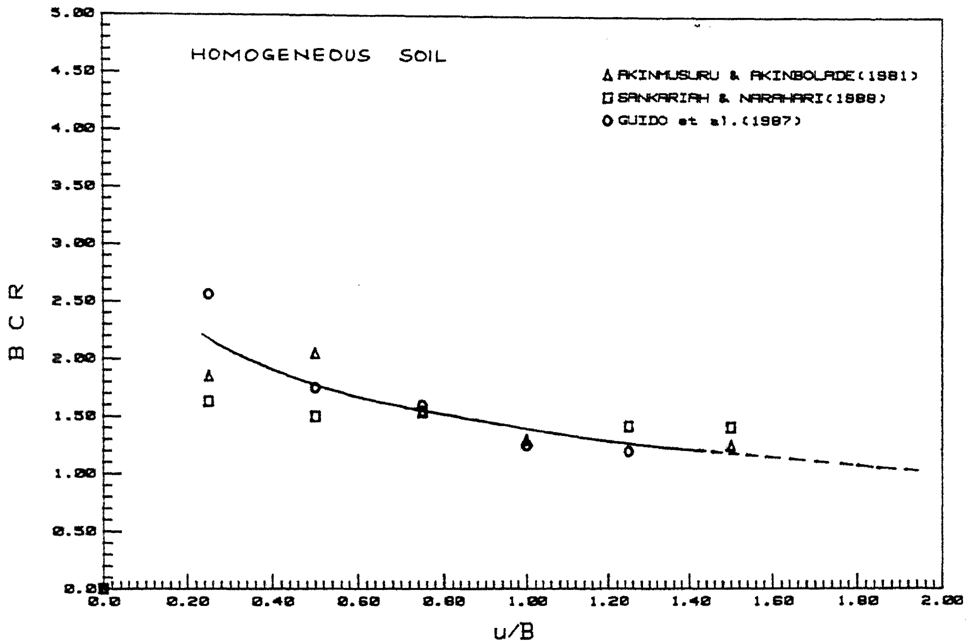


Fig.3.5 EFFECT OF u/B RATIO ON BCR (SQUARE FOOTING)

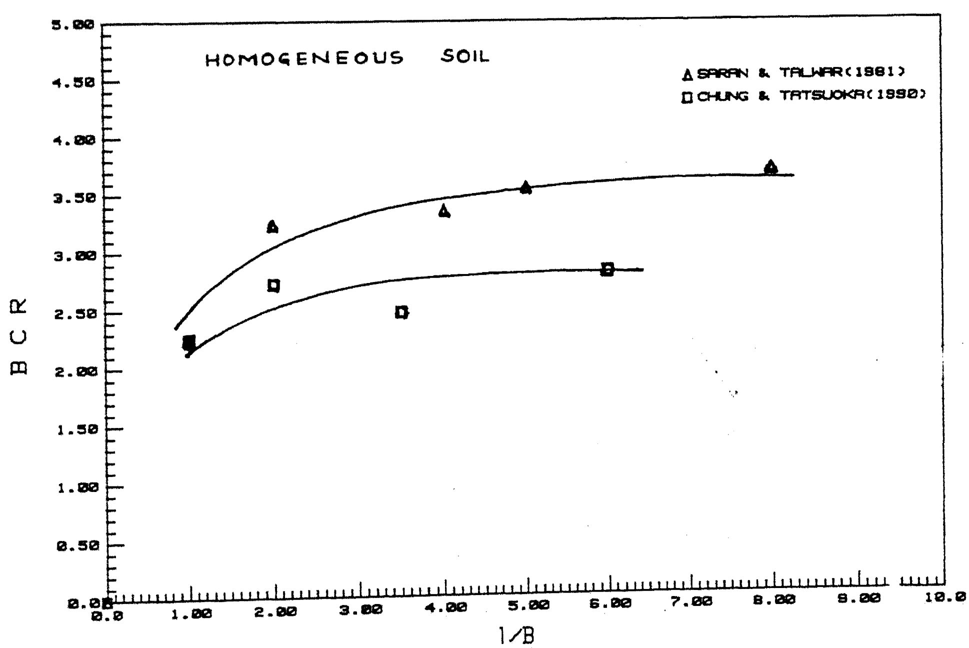


Fig.3.6 EFFECT OF l/B RATIO ON BCR (STRIP FOOTING)

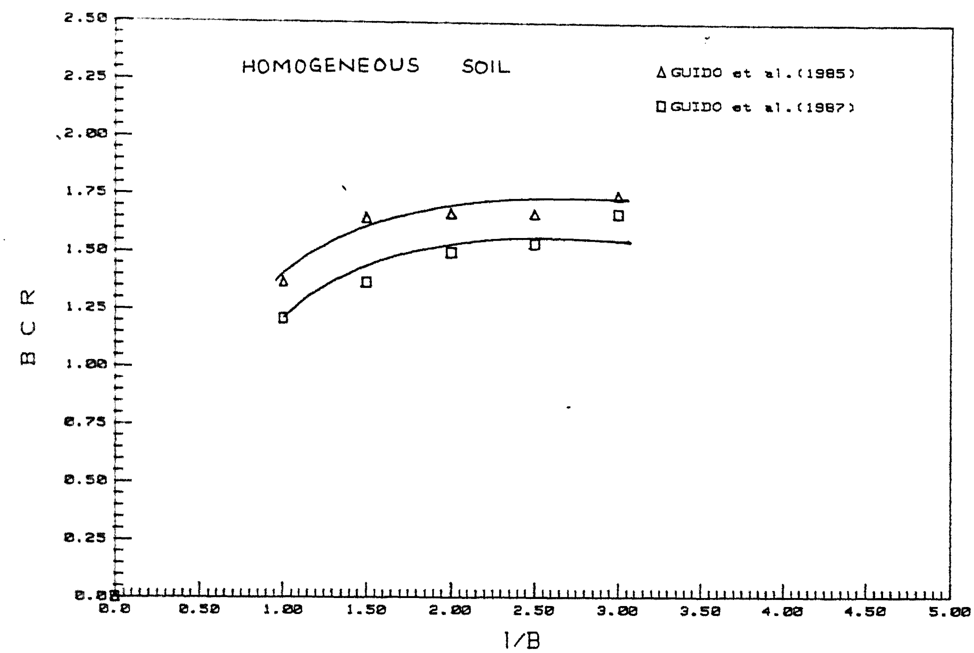


Fig.3.7 EFFECT OF l/B RATIO ON BCR (SQUARE FOOTING)

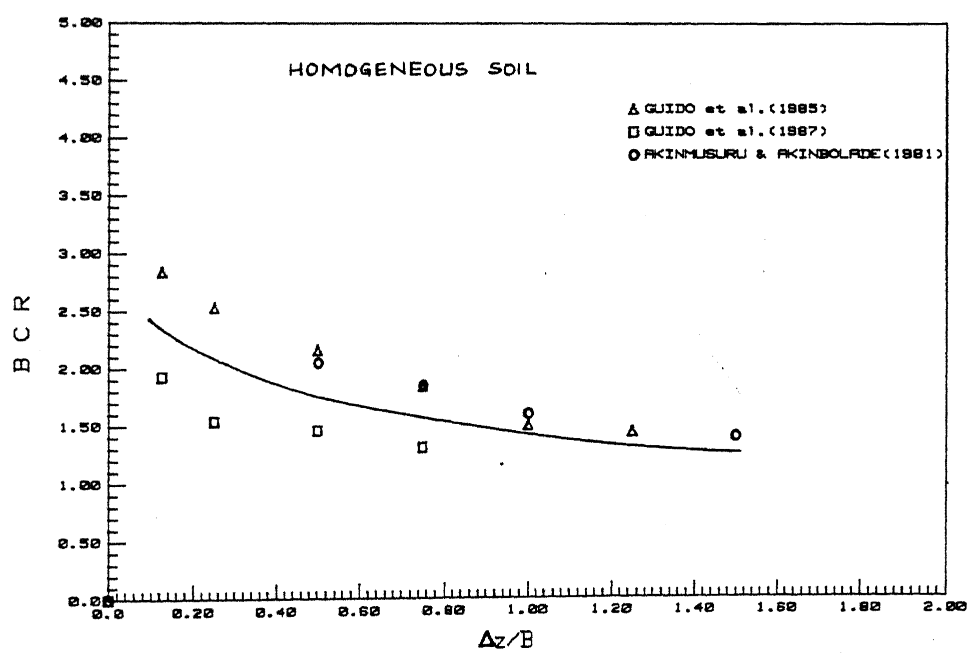


Fig.3.8 EFFECT OF Δz/B RATIO ON BCR (SQUARE FOOTING)

appreciable. The improvement in BCR is small being 50% compared to 200% in case of strip footings.

(iv) Effect of Vertical Spacing ($\Delta z/B$) of the Reinforcement

The effect of vertical spacing ratio ($\Delta z/B$) on BCR is shown in Fig.3.8 for square footings. It can be observed that as the spacing ratio increases the BCR decreases initially and reaches a almost constant value of 1.5 for ($\Delta z/B$) ratio greater or equal to unity. This is due to the fact that with further increase of $\Delta z/B$, there is little interaction between the lower layers of reinforcement with the failure zone of footing.

3.3.2 GRANULAR LAYER OVER SOFT SOIL AND HORIZONTAL REINFORCEMENT

It is well known that reinforced foundation beds contribute to an increase in the stiffness and the bearing capacity of the foundation -reinforced granular fill-soft soil system. The various components and parameters that affect these improvements are

(i) Granular Fill- its thickness, relative density and angle of shearing resistance

(ii) Reinforcement- Type, its location, spacing and length

Their contributions are evaluated and collated in these following paragraphs.

(i) Effect of Depth of Granular Fill

In most practical situations, a granular fill is laid over soft clays. Jarret(1986), Love et al.(1987) and Dembicki and Alenwicz(1988) present load settlement curves for a granular layer over a soft soil. Based on their results, the modular ratio, M_r,

defined as

$$MR = \frac{E_{gs}}{E_s}$$

where E_{gs} and E_s are the equivalent modulus of deformation of the granular fill-soft soil system, and of the soft soil alone respectively.

From Fig.3.9 it can be seen that for a strip footing MR (as calculated at 10% s/B, where s is settlement and B is breadth of footing) increases with an increase in the thickness of the granular bed .The general trend of the studies shows a linear increase of MR with the increase of depth of granular fill for thickness ratio (H/B) less than 1.5.

(ii)Effect of Reinforcement(Provided at the Interface between the Soil and Granular Fill)

Fig.3.10 shows the variation of the additional contribution to MR (ΔMR) due to the placement of a reinforcement layer at the interface between granular fill and soil with the thickness ratio (H/B) of the overlaying granular bed. It can be seen from this figure that the variation of the additional increase in the modular ratio due to the placement of the reinforcement layer is constant with H/B ratio. Thus the contribution of reinforcement to the increase in MR is independent of the granular layer thickness.

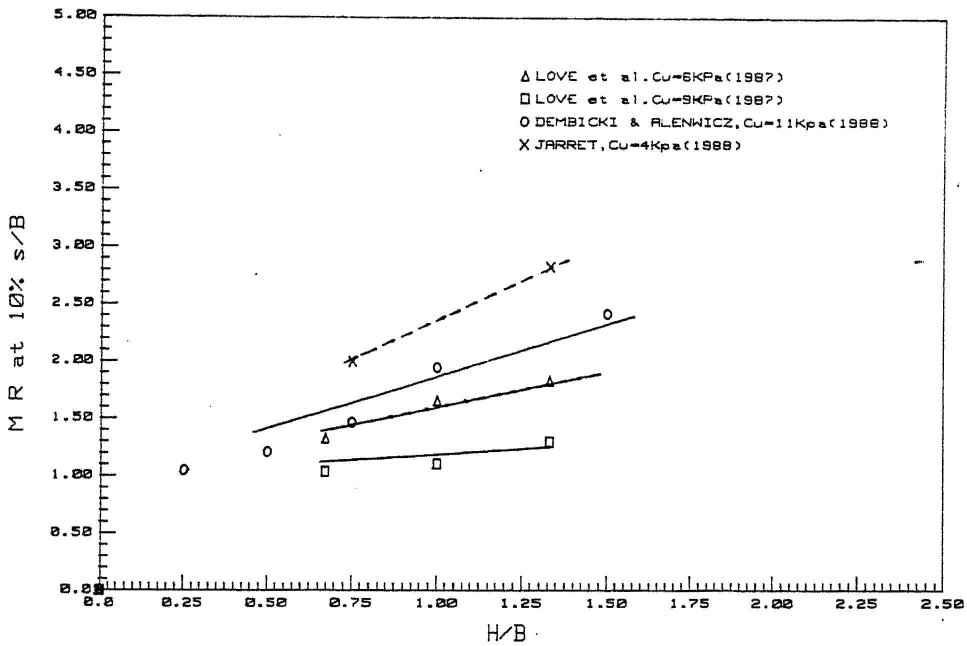


Fig. 3.9 EFFECT OF THICKNESS RATIO (H/B) OF OVERLAYING GRANULAR BED ON MR (STRIP FOOTING)

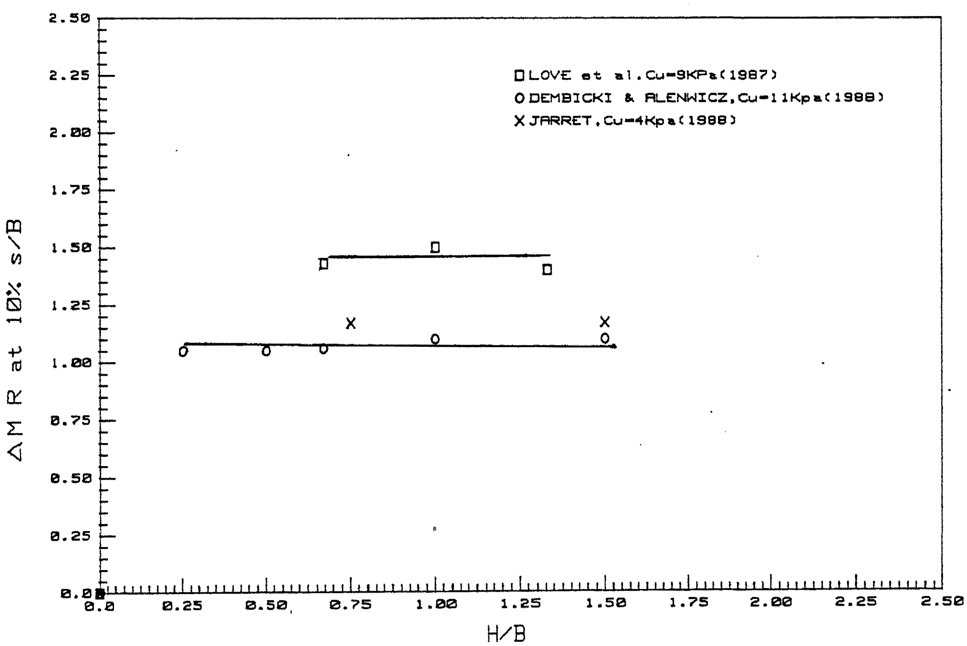


Fig.3.10 Addtional Improvement in MR(by placing a geotextile layer at interface)-ΔMR vs H/B

3.4 CONCLUSIONS

From the above discussions on the various test results on reinforced foundation beds, the following conclusions can be drawn.

1. With the increase in the number of reinforcing layers

(a) BCR increases, which reaches a limiting value after 6 to 7 layers of reinforcement.

(b) MR increases at an increasing rate.

2. With the increase in the depth to the first layer of reinforcement, BCR decreases and reaches a value of 1.0 for u/B=1.0.

3. With the increase in the length of the reinforcement, BCR increases upto l/B = 6 to 8, after which the increase is comparatively very small.

4. With the increase in the vertical spacing of the reinforcement, BCR decreases initially, but reaches a steady value of 1.5 at $\Delta z/B \geq 1.0$.

5.(a) Placing a granular fill layer over a soft soil improves its settlement behaviour considerably.

(b) Placement of a reinforcing layer at the interface between the soft soil and the granular fill, further improves the settlement behaviour.

CHAPTER IV

EXPERIMENTAL STUDY OF REINFORCED DOUBLE FACED WALLS

4.1 INTRODUCTION

Geotextiles and geogrids can be used for constructing reinforced embankments with comparatively steeper slopes as compared to conventional slopes. Thus the construction cost as well as the land needed for the purpose are reduced. The design of reinforced embankments involves the selection of type of geosynthetics, the spacing of reinforcing layers and the overlap between the free ends of the lower layer and the upper layer. In this chapter the latter two parameters are investigated for a vertical sided embankment or wall by model testing. The response of a reinforced embankment due to vertical loading over a part of its surface is studied through the measurement of vertical settlement of the loading plate and the variation of the lateral(horizontal) displacements with depth.

4.2.1 EXPERIMENTAL SETUP

The tests were conducted in a steel tank (Fig 4.1) 1.0m long, 0.3m wide and 0.5m deep. A perspex sheet 10mm thick was fitted on the front of the box, so that the horizontal deformations of the layers can be noted as indicated by the dial gauges. Model footings were made of 10mm thick mild steel plate of size 150mm×300mm and 100mm×300mm. The load was applied at the centre of the footing. The vertical settlement of the model footing was measured by placing two dial gauges at the central portion of the

B = WIDTH OF THE PLATE
Lo = OVERLAP
N = NUMBER OF LAYERS.

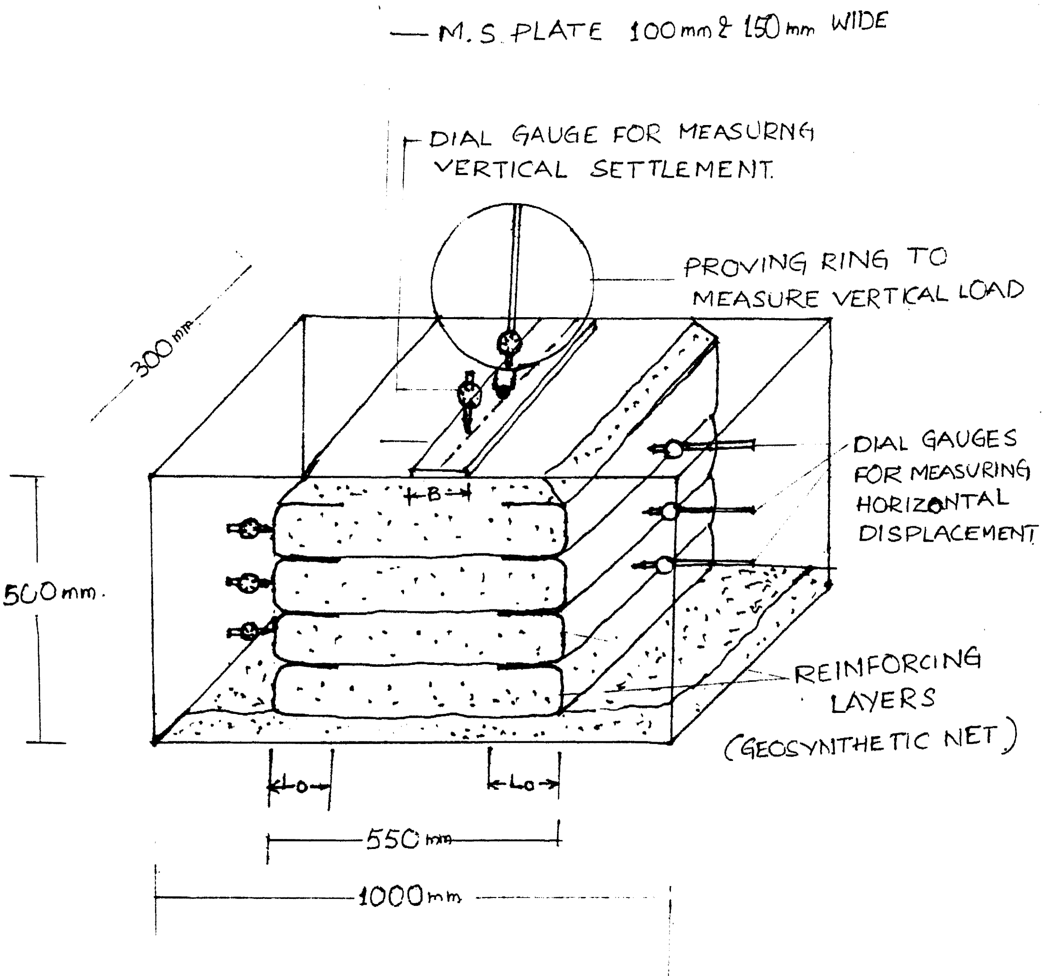


Fig. 4.1. SCHEMATIC DIGRAM OF THE EXPERIMENTAL SET UP

plate. The horizontal deformation of reinforced layers were measured by placing dial gauges at the mid height of each layer. The model embankment was constructed by initially placing two wooden planks on both side of the embankment and removing them after the embankment was fully prepared. The embankment was built up by pluviating air dried *Kalpi* sand from a hopper which is moved over the model box repeatedly. The height of fall was kept between 60 to 65cm and the pouring of 5Kg of sand took 3minutes and 30seconds. By this procedure homogeneous sand models having relative density in the range of 50 to 55% were obtained. At each prescribed depth, the pluvition was temporarily stopped, the lower synthetic layer was folded and a new layer of geosynthetic was placed on top in position. The procedure was repeated till the desired height of embankment was obtained.

4.2.2 SOIL

The sand used was obtained from *Kalpi* river with a mean size D_{50} of 0.40mm, coefficient of uniformity Cu of 2.08 and a coefficient of curvature Cc of 0.82. The minimum and maximum dry densities were γ_{dmin}=14.8KN/m^3, γ_{dmax}=16.0KN/m^3. The dry unit weight at which the experiments were conducted was γ_{d50}=15.78KN/m^3 at a relative density of D_r=50%. The angle of shearing resistance of the sand as obtained by direct shear test was 42^o at D_r=50%.

4.2.3 GEOSYNTHETIC

The geosynthetic used is produced and marketed by Netlon and is commercially avaible as screen.The tensile strength of the geosynthetic net as determineu from the test is 2.5KN/m width at

50% strain.

4.2.4 TEST PROCEDURE

After the reinforced embankment was built up, the top surface was leveled and the steel plate was placed on top of the model embankment. Two sizes of steel plates viz. 100mm×300mm and 150mm×300mm were used for the tests. Two dial gauges to measure the vertical and three dial gauges on each side of the embankment to measure the horizontal deformations were then placed in position. The vertical load was applied through a loading frame fitted with a proving ring. Reading were taken of the vertical load, vertical and horizontal displacements of the soil, at regular intervals.

4.3 RANGE OF PARAMETERS

The response of a 500mm high reinforced wall(embankment with vertical side walls) is investigated for the following range of parameters.

No. of reinforcement layers - 3,4,5,7

(centre to centre spacing of layers-130mm, 100mm, 80mm, 60mm.)

Overlap length L_o= 100mm, 50mm, 200mm

Width of loading plate- 100mm, 150mm.

4.4 RESULTS AND DISCUSSIONS

The response of the footing resting on the top of a vertical double faced reinforced wall has been studied for the range of parameters presented in the previous section. The observed results are presented in the form of stress-settlement diagrams in Fig.4.2 to 4.6. From these diagrams the ultimate bearing stresses (q_{max})

for the various cases have been estimated wherever possible. Values of initial tangent modulii have also been calculated. They are summarised in Table 4.1.

Table 4.1. Ultimate Bearing Pressure and Initial Tangent Modulus Values of all the Tests

		B=100mm			B=150mm	
	N	Lo=200mm	150mm	100mm	Lo=200mm	150mm
q_{max} (KPa)	3	16.4	15.1		34.2	28.3
	4	90.2	64.2	54.4	-	-
	5	100.5	95.5	83.1	-	-
	7					
K_s (KN/m^3)	3	133.0	100.0		121.0	95.0
	4	165.0	140.0	119.0	154.0	135.0
	5	190.0	180.0	175.0	180.	167.
	7	322.	311.	-	-	-

The effect of the various factors like the number of reinforcing layers, overlapping length of the reinforcement and the width of the footing on the ultimate bearing stress and the deformation behaviour have been discussed under separate headings as follows :

4.4.1 Effect of Number of Reinforcement Layers

Fig.4.2 shows the stress-vertical settlement curve for a 100mm wide plate for N=3, 4 and 5 and with an overlap length of reinforcement as 200mm. It can be observed that when the number of layers is 3, the maximum load taken by the embankment is only

15KPa, beyond which the rate of increase of lateral and vertical displacements were high indicating collapse. But the response of the reinforced wall improved significantly for four and five layers of reinforcement (the spacing of the layers were 100mm and 80mm respectively). The stress-settlement response for four and five layers of reinforcement for a loading plate of width 100mm is similar to general shear failure type of response. The curves for N=4 and 5 shows respectively peak stresses of 90.2 and 100.5KPa at a displacement of 10mm. If the number of reinforcement layer is 7 settlements are less for all stresses. The tendency of flattening of the stress- displacement curve shows that the stress level is very near to the failure stress. The embankment could not be loaded further due to the limitation of the loading system. The response of the reinforced earth wall is similar for other values of the overlapping lengths, 150mm and 100mm (Fig. 4.3 & 4.4) and for a wider footing (B=150mm) with overlap length of 200mm (Fig.4.5) and 150mm (Fig.4.6).

Fig.4.7 represents the variation of the ultimate bearing capacity of the embankment with the number of reinforcement layers. It can be observed from this figure that there is an increase in bearing capacity when the number of layers is increased. The curve is concave downward indicating that the rate of increase in bearing capacity decreases with the increase in the number of layers.

Fig.4.8 shows the variation of initial tangent modulus with the number of reinforcement layers. The curves indicate that K_s increases at an increasing rate with the increase of reinforcement layers.

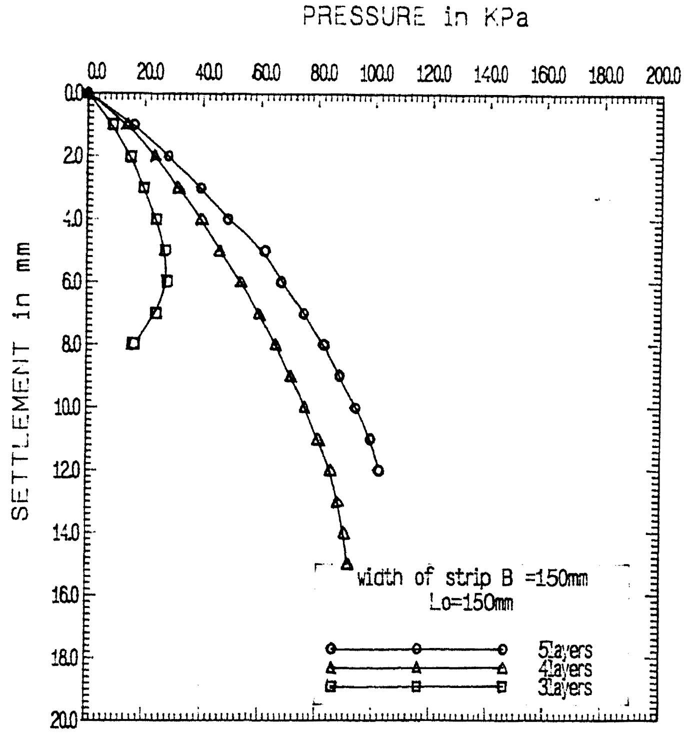


FIG. 4.3 PRESSURE VS SETTLEMENT RELATION FOR A STRIP OF 150 mm WIDE WITH Lo=150mm FOR DIFFERENT NUMBER OF LAYERS

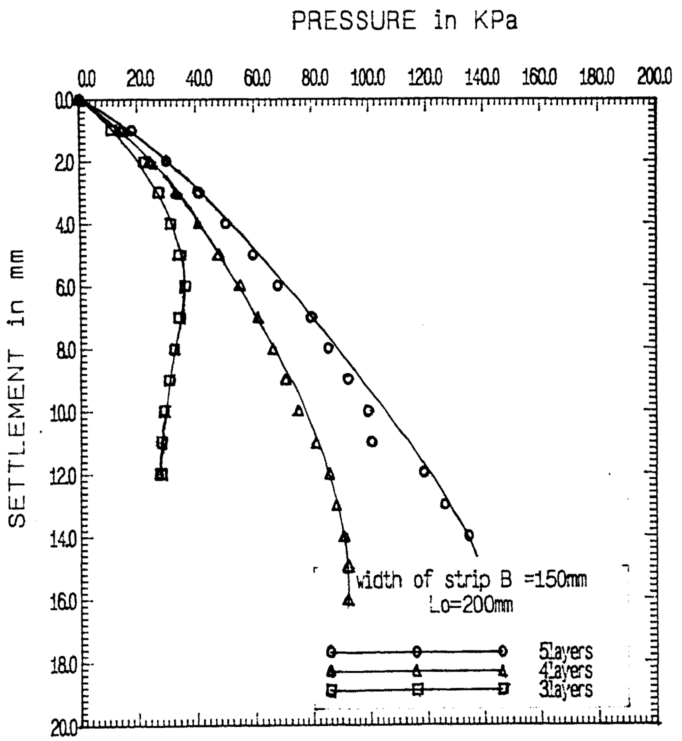


FIG. 4.2 PRESSURE VS SETTLEMENT RELATION FOR A STRIP OF 150 mm WIDE WITH Lo=200mm FOR DIFFERENT NUMBER OF LAYERS

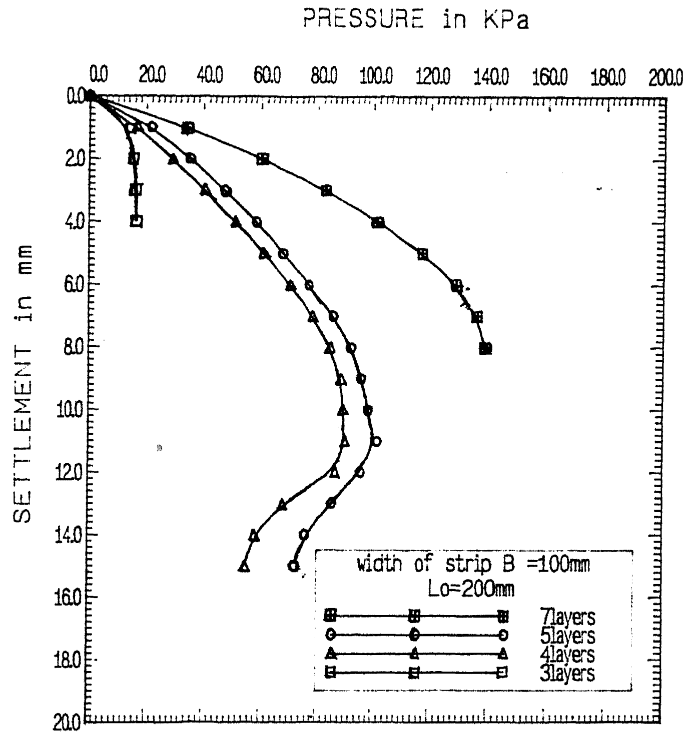


FIG. 4.4 PRESSURE VS SETTLEMENT RELATION FOR A STRIP OF 100 mm WIDE WITH Lo=200mm FOR DIFFERENT NUMBER OF LAYERS

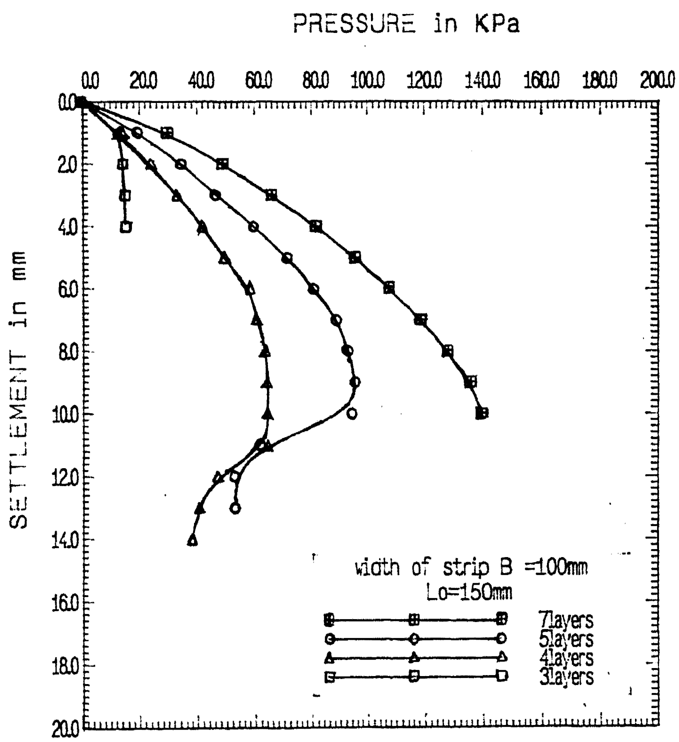


FIG. 4.5 PRESSURE VS SETTLEMENT RELATION FOR A STRIP OF 100 mm WIDE WITH Lo=150mm FOR DIFFERENT NUMBER OF LAYERS

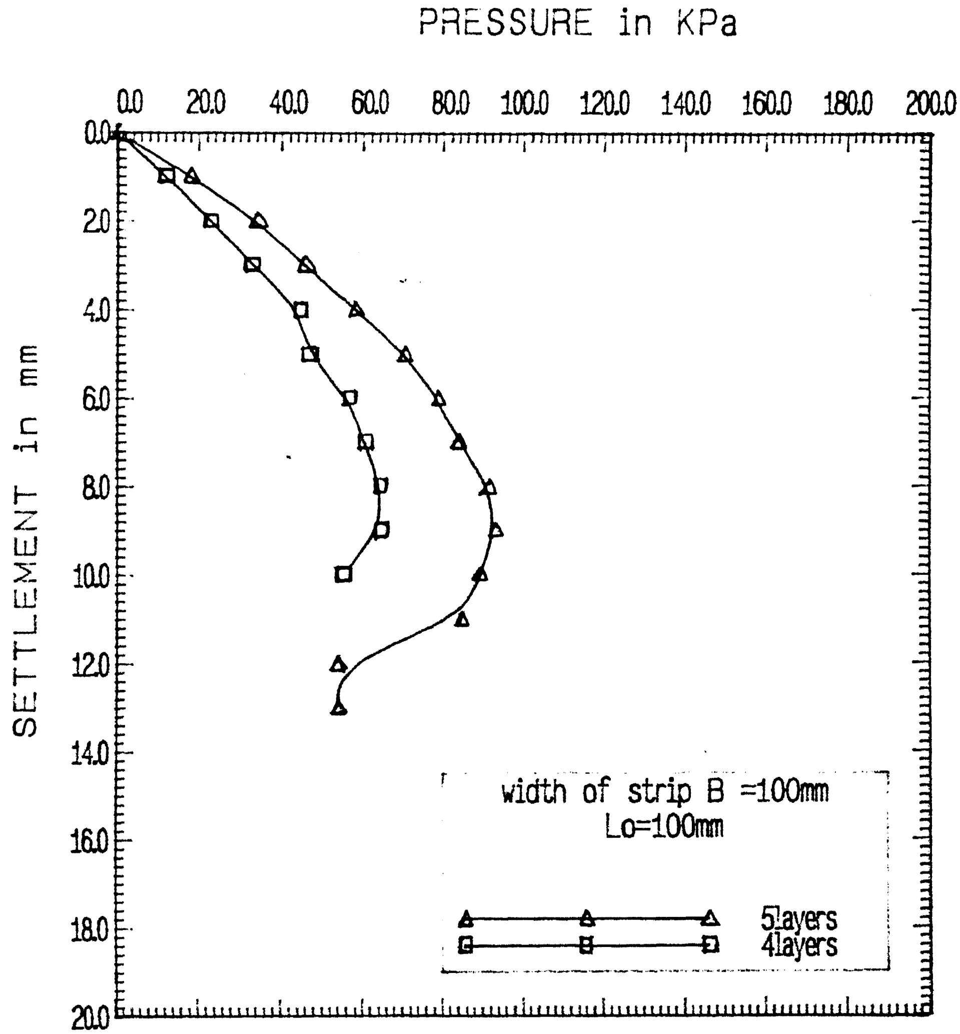


FIG. 4.6 PRESSURE VS SETTLEMENT RELATION FOR A STRIP OF 100 mm WIDE WITH Lo=100mm FOR DIFFERENT NUMBER OF LAYERS

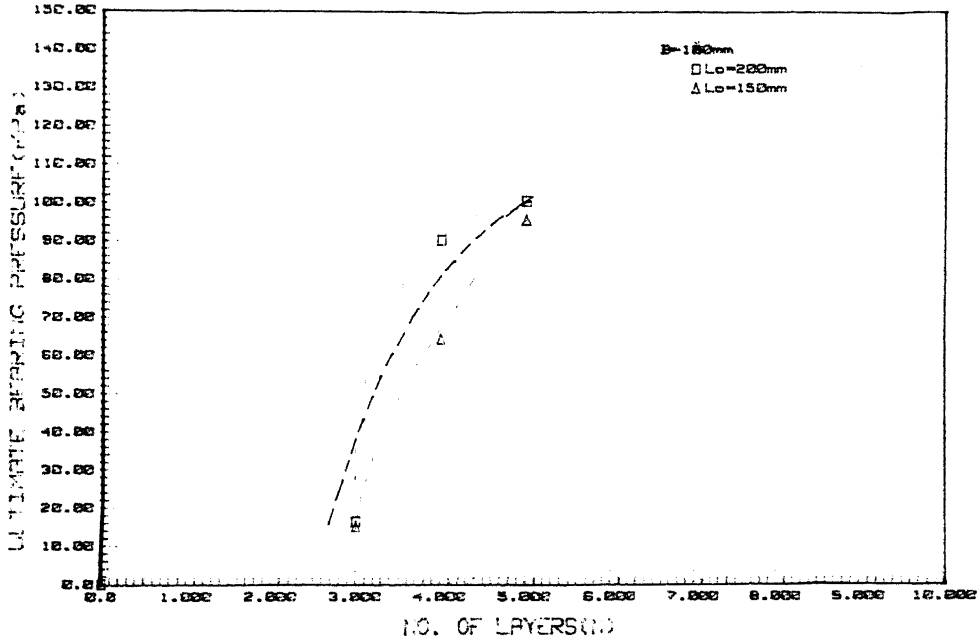


Fig. 4.7. Effect Of No. of Layers (N) on Ultimate Bearing Pressure

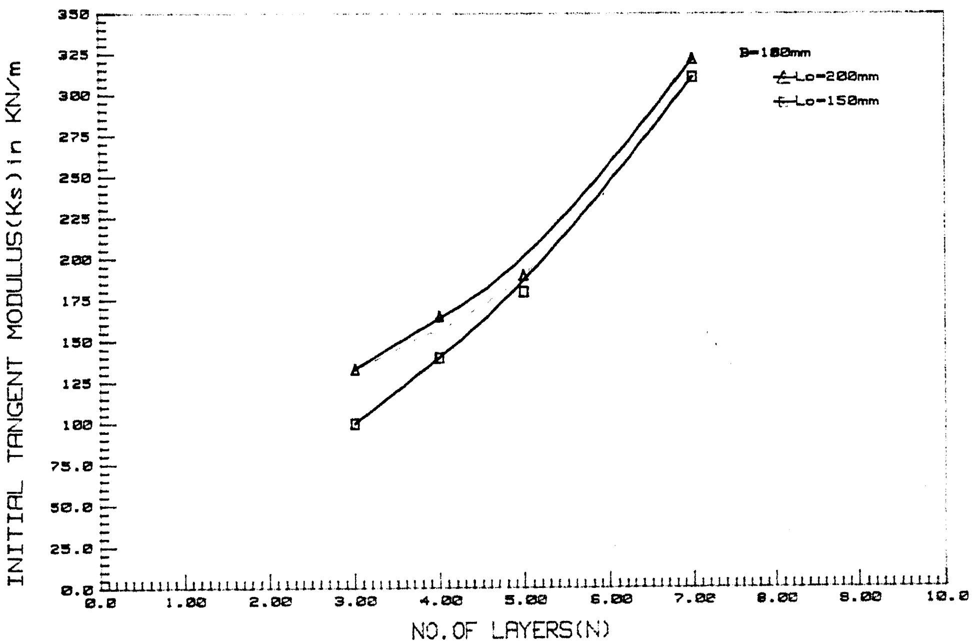


Fig.4.8 Effect of No. of Layers(N) on

Initial Tangent Modulus

The variation of lateral displacements with depth with increase in applied stress/settlement are presented in Fig. 4.9 to 4.11. It is seen that with an increase in number of reinforcing layers the lateral deflections at the top of the wall decrease but the same at greater depths tend to increase.This can probably be explained as follows:

The reinforcing layers impart more rigidity in the lateral direction than in the vertical direction. So with the increase in the number of layers the rigidity in the lateral direction increases proportionately. As a consequence the wall face tries to deform more uniformly manifesting in larger deflections at lower depths.

Fig. 4.12 represents the curve for S_v versus S_h, where S_v is the settlement and S_h is the corresponding horizontal displacement at the top layer for 4, 5 and 7 numbers of reinforcement layers for a strip of 100 mm width and overlap of 150 mm. From this figure it can be noted that the horizontal displacements of the top layers at the initial stages of loading are nearly same but as the load increases the curves deviate resulting in larger vertical displacements. The difference increases towards the failure point and the embankment with the highest number of layers shows the least horizontal displacement.

4.4.2 Effect of Length of Overlap

Fig.4.13 shows the effect of the length of overlap on the ultimate bearing capacity of the reinforced embankment. It can be observed that the ultimate bearing capacity increases linearly with the increase in the length of the overlap

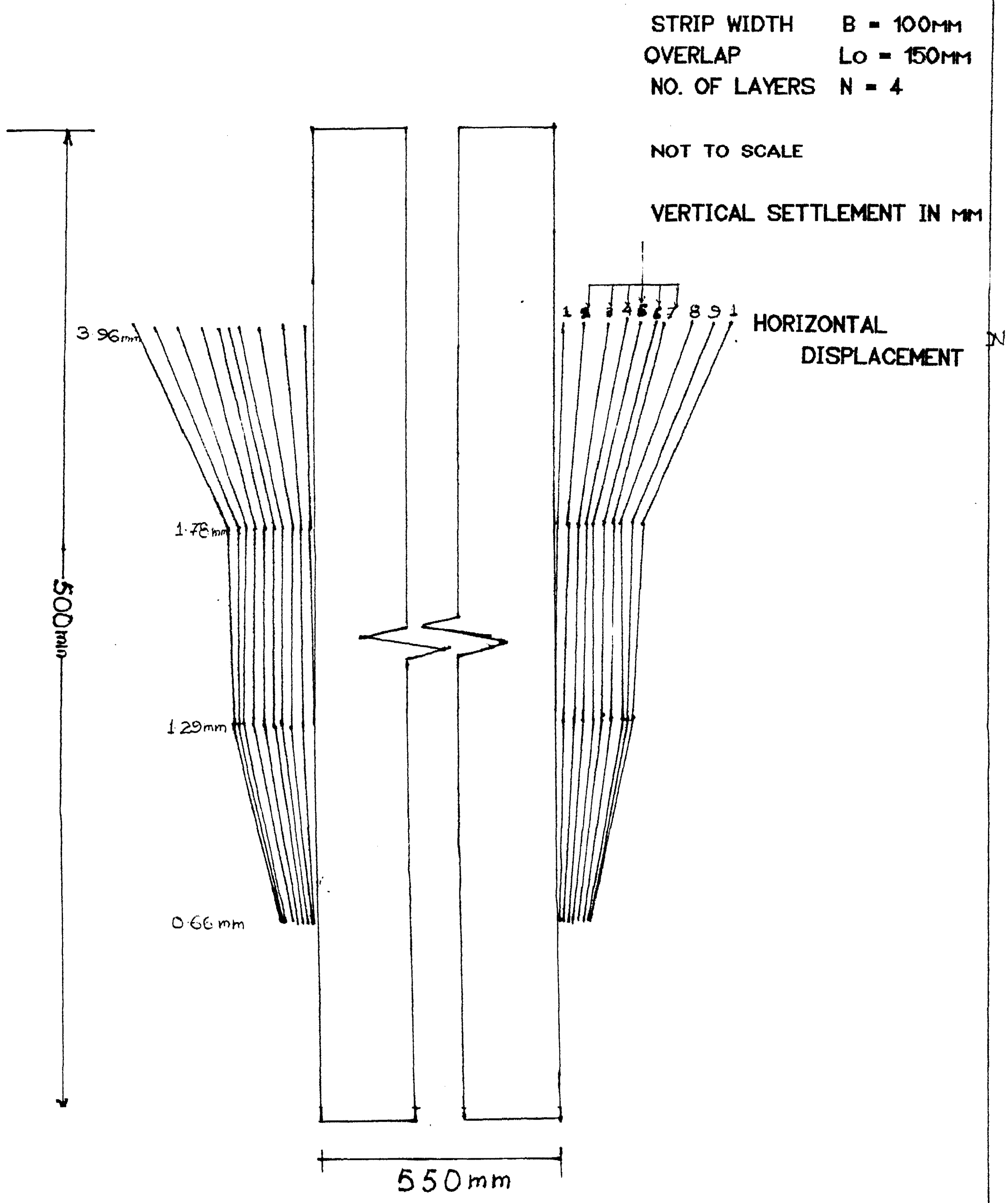


FIG.4.9 LATERAL DISPLACEMENTS WITH DEPTH WITH INCREASE IN APPLIED STRESS/SETTLEMENT FOR B=100MM, Lo=150MM & N=4

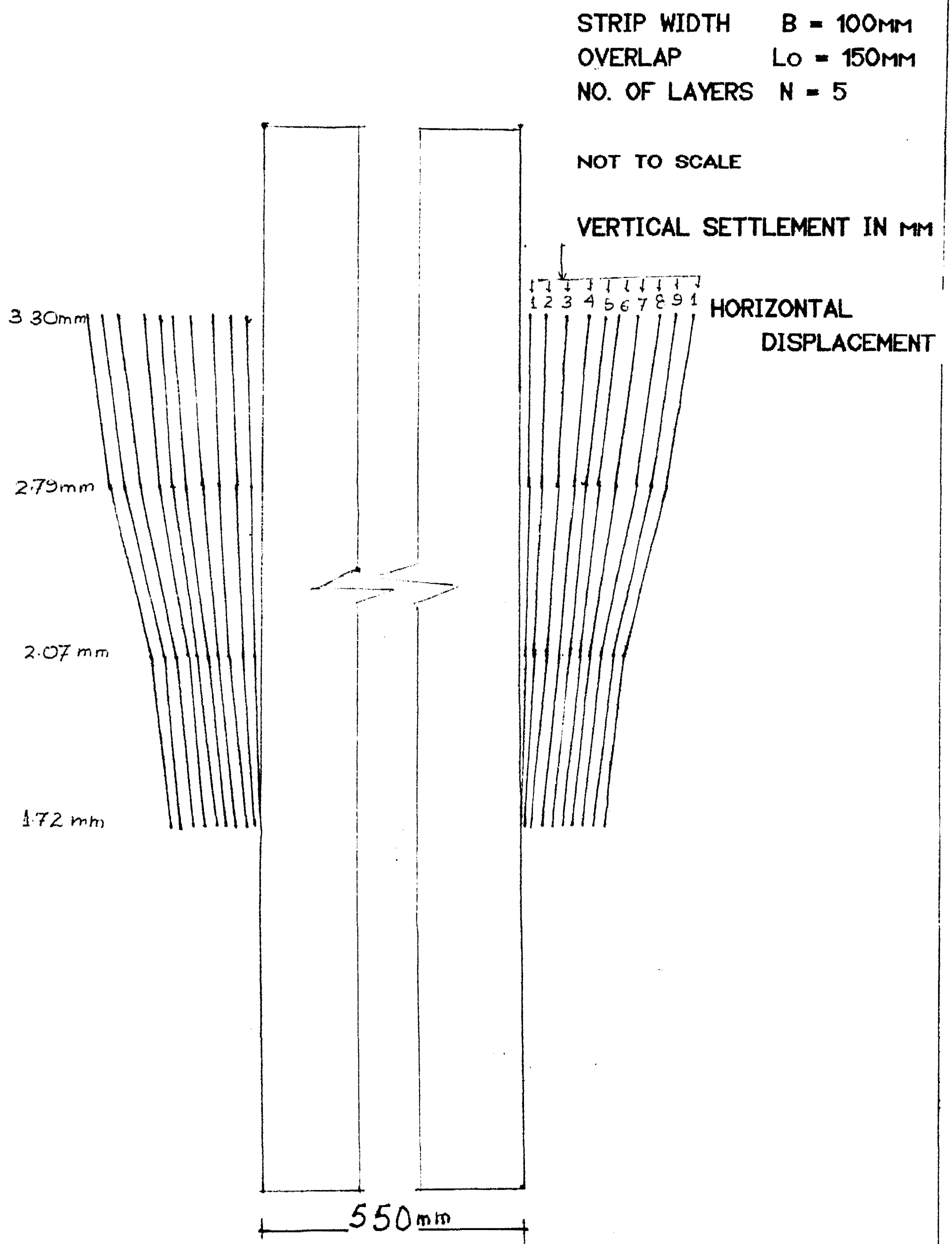


FIG.4.10 LATERAL DISPLACEMENTS WITH DEPTH WITH INCREASE IN APPLIED STRESS/SETTLEMENT FOR B=100MM, Lo=150MM & N=5

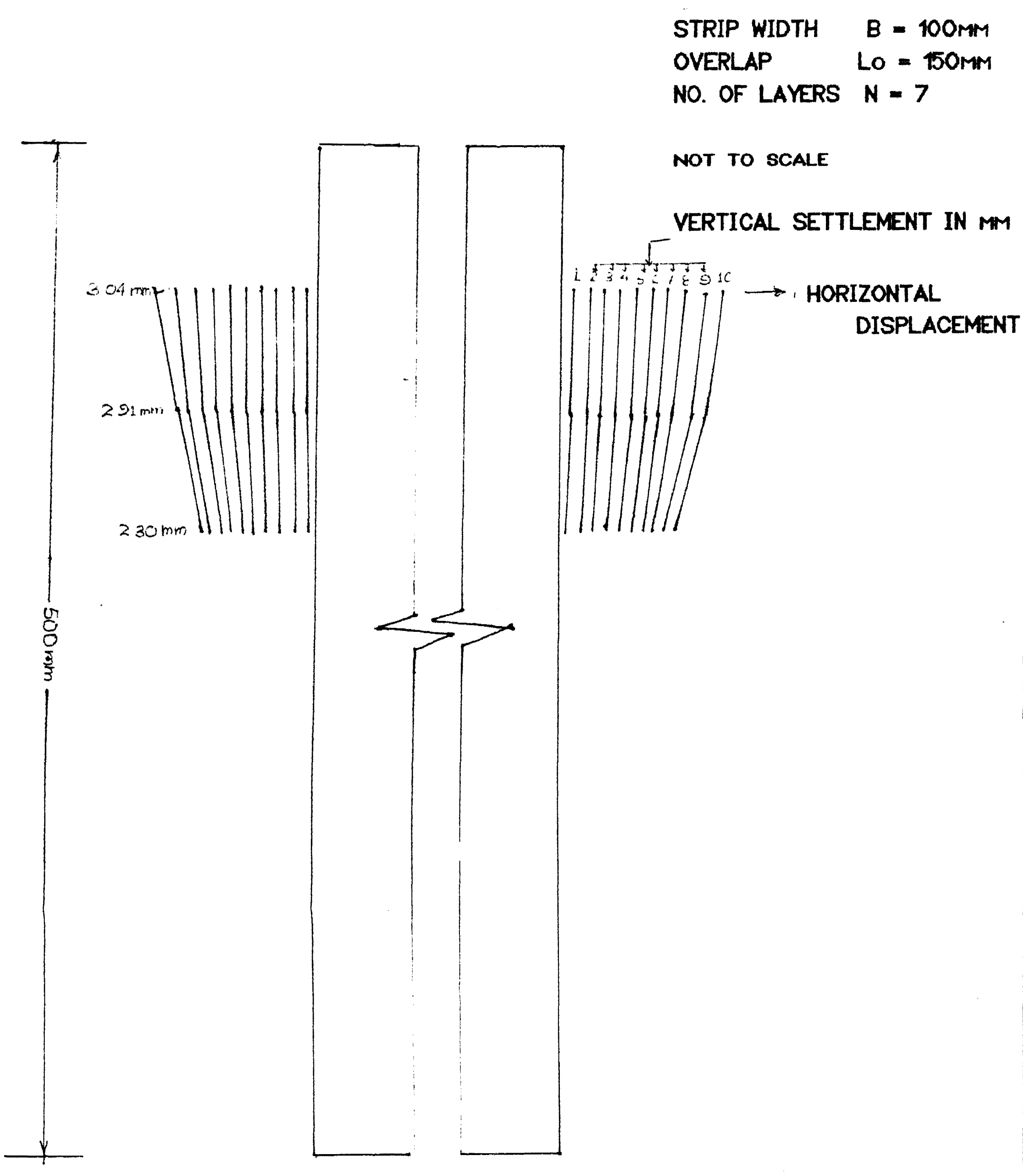


FIG.4.11 LATERAL DISPLACEMENTS WITH DEPTH WITH INCREASE IN APPLIED STRESS/SETTLEMENT FOR B=100MM, Lo=150MM & N=7

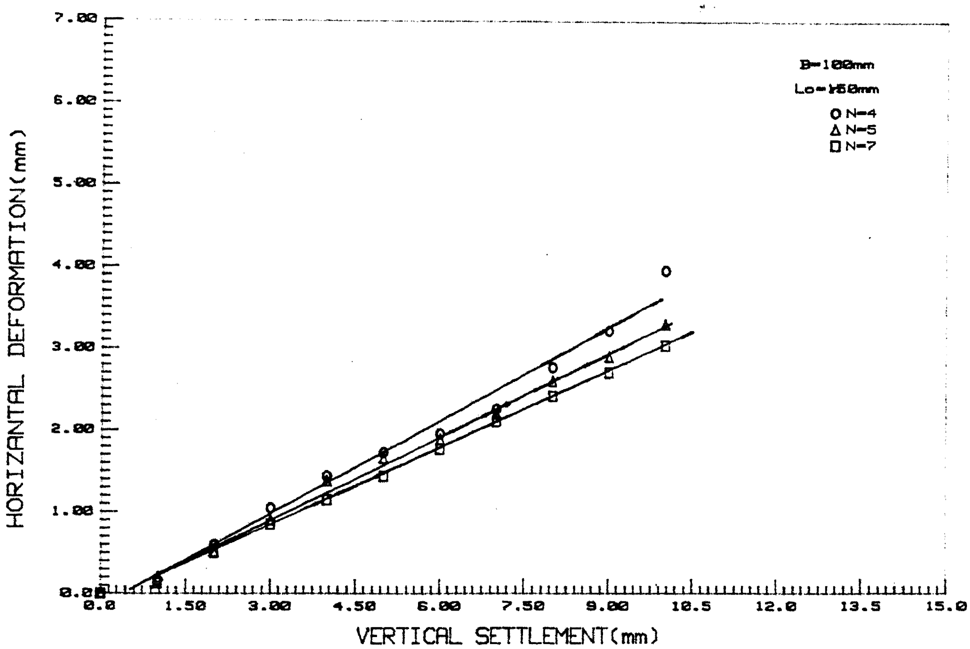


Fig.4.12 Vertical Settlement vs Horizantal Settlement

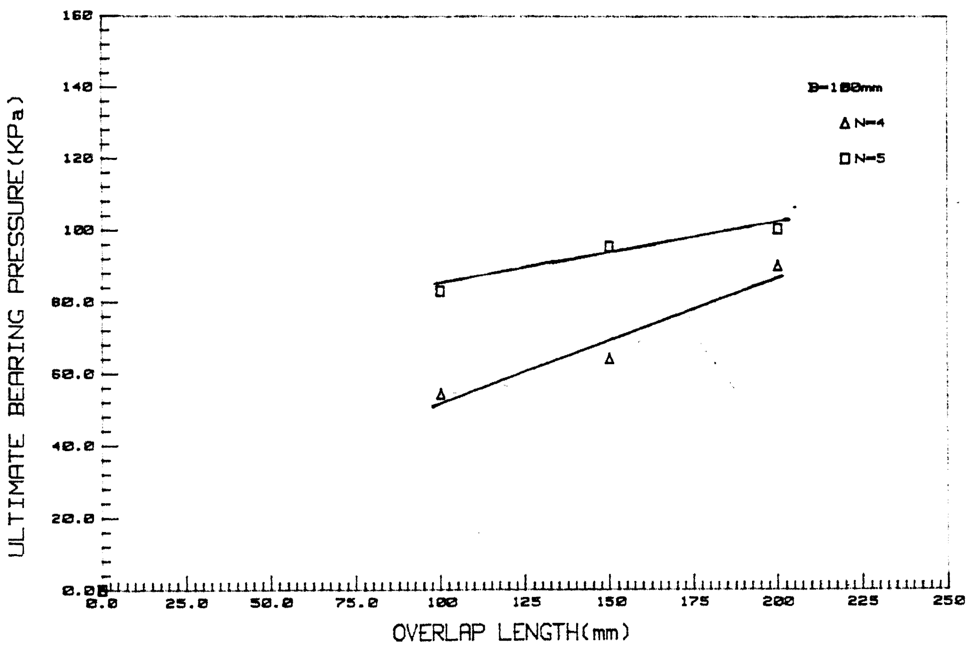


Fig 4.13 Effect of Overlap.Length(Lo) on

Ultimate Bearing Pressure

The effect of the length of the overlap on the initial tangent modulus is shown in Fig.4.14. It can be observed that K_s also increases linearly with the increase of the overlap length.

4.4.3 Effect of Width of Plate

The effect of width of the plate on the initial tangent modulus has been estimated by comparing the computed values of the initial tangent modulus obtained from the observed stress displacement diagrams (Fig 4.2 to 4.6) for different width of footing and is presented in Table 4.2.

Table 4.2. Effect of Width and Number of Reinforcing Layers on Ks

N	K_s (KN/m^3)		
Lo=200mm	B=100mm	B=150mm	% decrease
3	133	121	9
4	165	154	6.7
5	190	180	5.2
Lo=150mm			
3	100	95	5
4	140	135	3.5
5	180	175	2.7

It can be observed that for a given overlapping length of the geotextile and number of layers, the initial tangent modulus decreases with an increase in width of the footing. The decrease is lesser for the embankment having more number of reinforcing layers. For the parameters studied the decrease in initial tangent modulus is within the range of 2.7% to 9%.

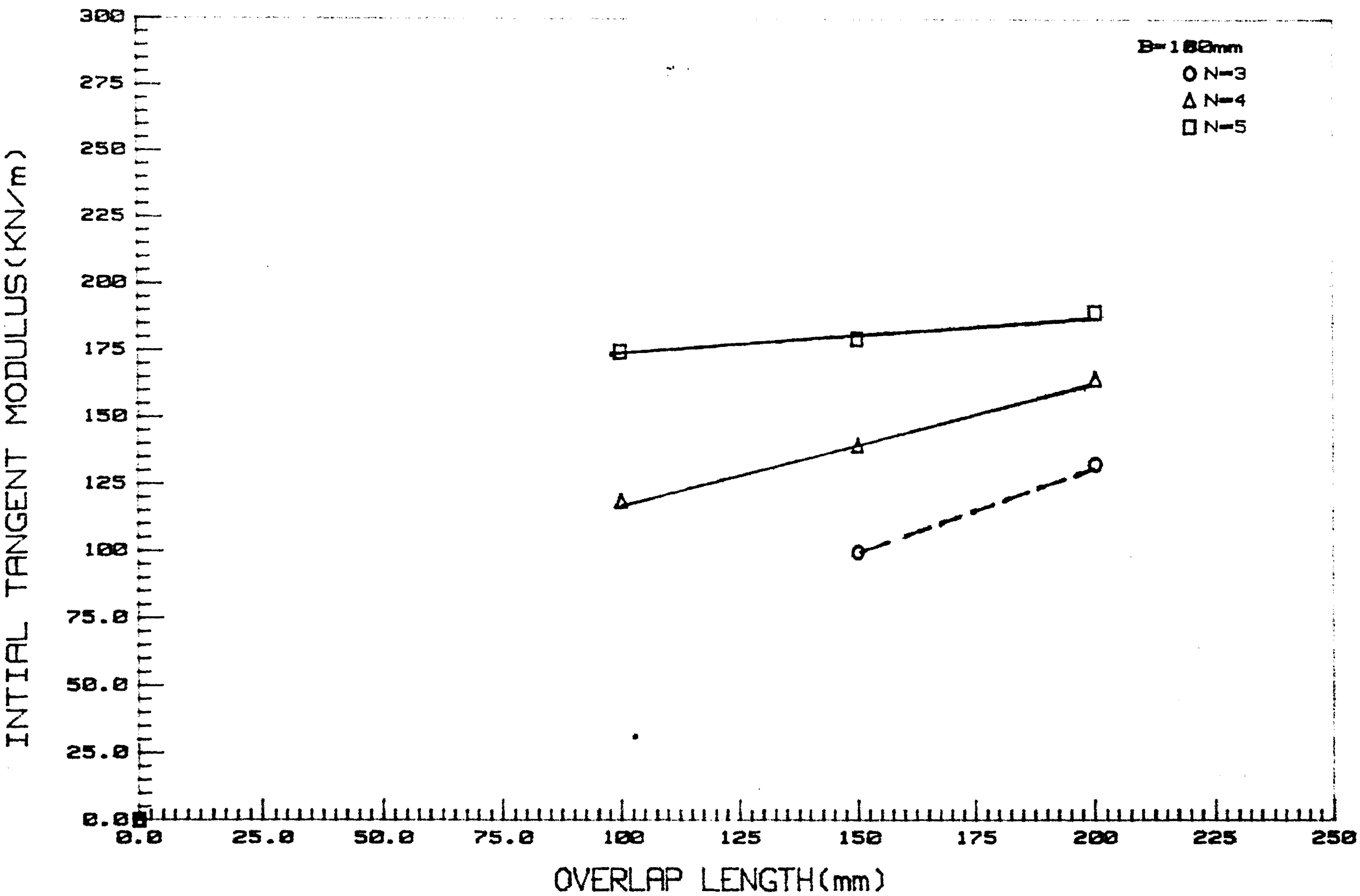


Fig.4.14 Effect of Overlap Length(Lo) on

Initial Tangent Modulus

4.5 CONCLUSIONS

Based on the above results and discussions the following general conclusions can be drawn:

1. With the increase in the number of reinforcing layers

(a) Bearing capacity increases.

(b) Initial tangent modulus increases with increasing rate.

(c).Lateral deflection at the top wall decreases.

2. The ultimate bearing capacity increases linearly with the increase in the length of the overlap.

3. For a given overlapping length and number of reinforcing layer the initial tangent modulus decreases with the increase in plate width.

4.6 SCOPE OF FUTURE STUDIES:

1 Studies considering the effects of more number of reinforcing layers and different reinforcing material on the stability and compressibility aspects in both cohesive and cohesionless soil.

2 Estimation of the ultimate bearing capacity of the reinforced beds and walls and the deformation pattern of the composite material using finite element methods and its comparison with the experimentally observed values.

3. Studies of the dynamic response of reinforced beds, walls and embankments.

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